

DOKUZ EYLÜL UNIVERSITY
GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES

**DEVELOPMENT OF AN EFFICIENT SEISMIC
RETROFITTING TECHNIQUE FOR TYPICAL
OLD PRECAST RC FRAMED BUILDINGS IN
ALBANIA**

by
Anila XHAHYSA

September, 2020
İZMİR

**DEVELOPMENT OF AN EFFICIENT SEISMIC
RETROFITTING TECHNIQUE FOR TYPICAL
OLD PRECAST RC FRAMED BUILDINGS IN
ALBANIA**

**A Thesis Submitted to the
Graduate School of Natural and Applied Sciences of Dokuz Eylül University
In Partial Fulfillment of the Requirements for the Degree of Doctor of
Philosophy in Civil Engineering**

**by
Anila XHAHYSA**

**September, 2020
İZMİR**

Ph.D. THESIS EXAMINATION RESULT FORM

We have read the thesis entitled “**DEVELOPMENT OF AN EFFICIENT SEISMIC RETROFITTING TECHNIQUE FOR TYPICAL OLD PRECAST RC FRAMED BUILDINGS IN ALBANIA**” completed by **ANILA XHAHYSA** under supervision of **PROF. DR. SERAP KAHRAMAN** and we certify that in our opinion it is fully adequate, in scope and in quality, as a thesis for the degree of Doctor of Philosophy.

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This study is dedicated to all the old civil engineers in Albania, whose work was precise in every step. The evaluation of the structures they designed is of great responsibility, and does not aim to find any flaws but bring these structures to the nowadays level of engineering science filter.

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Anila XHAHYSA

DEVELOPMENT OF AN EFFICIENT SEISMIC RETROFITTING TECHNIQUE FOR TYPICAL OLD PRECAST RC FRAMED BUILDINGS IN ALBANIA

ABSTRACT

This study aimed to evaluate the seismic performance of a typical of RC precast structure that was designed for the highest seismic region in Albania. The high seismic activity of the country, the deficiencies of old design codes and seismic maps and the considerable number of reinforced concrete precast structures that were designed prior to the aseismic regulations came to force, highlight the importance of this study.

An optimal seismic strengthening alternative, referring to the observed deficiencies in these structures is suggested. Specifically, an alternative brace characterized by significant energy dissipation and early activation properties is developed. Improvement in terms of strength, stiffness and ductility without requiring foundation strengthening, confirms the suggested alternative as the optimal strengthening scenario.

The study is further extended by creating an enhanced version of the alternative brace, in order to produce minimal residual deformations and to be applicable in new structures as well that target a high seismic performance.

Keywords: Reinforced concrete, Albania, strengthening, brace, UFP Plate, Performance

ARNAVUTLUK'TA BULUNAN TİPİK BETONARME ÖNÜRETİMLİ ÇERÇEVELİ BİNALAR İÇİN ETKİN BİR SİSMİK GÜÇLENDİRME TEKİNİĞİNİN GELİŞTİRİLMESİ

ÖZ

Bu çalışma Arnavutluk'taki en yüksek sismik bölge için tasarlanmış tipik betonarme prefabrik yapıların sismik performansını değerlendirmeyi amaçlamıştır. Ülkenin yüksek sismik faaliyeti, eski tasarım yönetmeliklerinin ve sismik haritaların eksiklikleri ve sismik yönetmelikler getirilmeden önce tasarlanan çok sayıda betonarme prefabrik yapıların varlığı bu çalışmanın önemini vurgulamaktadır.

Bu yapılardaki gözlenen eksikliklere dayanarak uygun bir sismik güçlendirme alternatifi önerilmektedir. Özellikle, kayda değer enerji dağılımı ve erken aktivasyon özellikleri ile nitelenen alternatif çaprazlı sistem geliştirilmiştir. Temel güçlendirmesi gerektirmeden, dayanıklılık, rijitlik ve süneklik parametrelerinin iyileştirmeleri, önerilen alternatif en uygun güçlendirme senaryosu olduğunu göstermektedir.

Bu çalışma, artık deformasyonların minimal düzeyde oluşması ve yüksek sismik performansı hedefleyen yeni yapılara da uygulanabilecek alternatif kendiliğinden merkezlenen çaprazlı sistemin geliştirilmiş bir versiyonunun oluşturulmasıyla daha da genişletildi.

Anahtar kelimeler: Betonarme, Arnavutluk, güçlendirme, çaprazlı sistem, UFP, Performans

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CHAPTER ONE

INTRODUCTION

1.1 Introduction

Albania is characterized by a relatively high seismic activity. If extended to regional terms, the seismicity of Balkan Peninsula is the highest in Europe. This is physically explained by the multiple interaction of the tectonic plates in the Aegean Sea and Adriatic Sea and the complicated tectonics in the Carpates (Aliaj, Koçiu, Muço & Sulstarova, 2010). The energy released from the earthquakes of this system arises to approximately 15% of the total energy released from all the earthquakes all over the world.

In this context, the design of structures in this country should carefully implement appropriate structural seismic provisions, in order to assure the proper seismic risk mitigation. Building regulations have been applied in Albania since 1952. The static method adopted in the first regulations for the seismic calculation of structures, was replaced in 1963 by a version of the dynamic method. Currently, Regulation KTP-N.2-89 is in force in Albania (KTP-N.2-89). However, recent studies have revealed some deficiencies that need to be revised even in this code. Research results (Aliaj et al., 2010) (UNDP Program, 2003) have shown that the last seismic risk assessments have produced higher seismic parameter values than preceding works.

For instance, a recent study on the seismic performance of masonry structures in Albania (Baballëku, 2014), provided a comparison between the elastic spectra generated based on the different seismic maps in the time frame 1963 - 1998, where it is quite visible the increase of spectral values in time. Hence, important modifications in the seismic demand should be implemented in the actual code itself.

In this context, the structures built prior to the current design code, despite having probable deficiencies in seismic detailing, were designed based in a non-conservative seismic demand as well. Referring to the statistical data on the building stock

(CENSUS, 2011) that are available in the Appendix A of this thesis, in total there are 187,633 residential buildings that were constructed before this design code was applied. This is a significant number of inhabited structures, that need an immediate seismic performance evaluation. As the main construction material of these structures was brick or stone, the research focused on seismic performance evaluation of masonry structures in Albania (Guri, Lluca & Luca, 2015; Korini & Bilgin, 2013; Paci & Bidaj, 2015). The general outcome of this research is that some of the structures designed with Albanian old codes can pose problems as a result of insufficient reinforcement detailing and a non-realistic seismic demand. The rest of structures that were composed of prefabricated reinforced concrete elements, wood or other material, was not properly evaluated though. However, a classification based on the building main construction material type of the residential buildings (CENSUS, 2011), shows that 10,594 prefabricated structures were designed before last seismic design code was implemented.

1.2 Thesis Aim and Scope

1.2.1 Aim

- Evaluation of the seismic performance of a typical of RC precast structure that was designed for the highest seismic region in the country, considering the high seismic activity of Albania, the deficiencies of old design codes and seismic maps, and the considerable number of reinforced concrete precast structures that were designed prior to the last seismic design code.
- Proposing an optimal seismic retrofit alternative, referring to the observed deficiencies in these structures.
- Development of an alternative brace characterized by significant energy dissipation and early activation properties and evaluation of its performance using FEM.
- Generation of the enhanced version of the alternative brace, in order to produce minimal residual deformations and to be applicable in new structures as well that target a high seismic performance and resilience.

1.2.2 Scope

The structure type selected for analysis in this study is a precast reinforced concrete frame structure, designed in 1982 for the highest seismic zones in Albania. This structure is a model that was designed to be constructed in different parts of the country that are prone to the highest seismic risk. Therefore, the seismic evaluation of this structure that is a “representative building type” is quite relevant. The performance of the structure in its original conditions and after being retrofitted, is analytically evaluated using SAP2000, 2007 Software.

The performance of the alternative brace generated in this study is evaluated using Finite Element Analysis (LS DYNA). An enhanced version of the brace is developed as well, so that minimal residual deformation is obtained. This improved version can be applied in new structures as well, that target a high seismic performance. The prestressing system, and the set of UFPs installed in the double-concentric-tube system is evaluated as well using the FEA (LS DYNA) approach.

1.3 Thesis Main Parts

Evaluation of the seismic performance of a typical of RC precast structure that was designed for the highest seismic region in the country and development of a new retrofit system that is adequate to the low ductility of the structure is presented in seven chapters as follows.

In the second chapter is provided a general review of the precast construction industry in Albania. The main features of the structural elements are described and the most used connection application methods are analyzed.

The third chapter focuses on the technical details of the precast frame chosen as the case study of this thesis. The structural details and the deficiencies present in the reinforcement detailing are presented in this section.

Chapter four, based on a probabilistic approach, is dedicated to produce the proper site seismic characteristics, as some deficiencies were observed in the old seismic maps used currently in the Albanian design codes.

In chapter five are presented the analytical procedures and the respective results for the seismic performance evaluation of the existing structure. It provides a clear view of the weak points of the frame during seismic excitation.

Chapter six is dedicated to the development of a new energy dissipating brace, to conform the retrofit requirements of the current frame. Advantages of this brace in terms of good redundancy and early activation, when compared to other braces, are analyzed with finite element method. Moreover, this section presents another improved version of the brace, in case application in new structures that require self-centering is needed.

In chapter seven are presented the results of the application of the retrofit strategy. Improvement in terms of strength, stiffness and ductility, confirms the suggested alternative as an optimal strengthening scenario.

Chapter eight provides a brief summary of the major outcomes of this thesis, with the respective recommendations and as well the points that need further research in the future.

Appendix A provides a statistical database for the number of buildings constructed in a specific period in Albania and their respective classification based on the building main construction material type.

Appendix B presents a table with all registered earthquakes that occurred in the period 1900-2005 (which are instrumentally recorded) located within the coordinates 41.87-42.31, 19.39-19.82 (Shkodra region).

Appendix C provides the detailed data for the column hinge generation, specifically moment-rotation and axial force-moment relationships are presented for each case.

Appendix D is dedicated to the finite element software analysis details. It shows specifically the reduced LS DYNA keyword for the mini-brace subjected to displacement-controlled loading and presents as well the main modeling stages of the specimen.

Appendix E provides the construction reinforcement details for the existing reinforced concrete frame representative elements, namely: column, beam and the respective joints. Precast hollow slab panels with the respective connections to form the composite unit are shown in detail as well.

Appendix F provides the necessary details for the innovative brace generated in this study. Dual tube system, UFP brace and enhancement of the brace by adding the tendons system are highlighted in this section.

CHAPTER TWO

REINFORCED CONCRETE PRECAST CONSTRUCTION IN ALBANIA

2.1 Structural Design in Albania and the Soviet Union Influence

Albania communist history of governance extended in the years 1946 to 1992 and was characterized by a typical Stalinist style in its administration. The authorities in that time introduced prefabricated building systems during 70s. A prefabrication plant was created and developed to meet the new urban housing demand. The first National State Institute of Studies, Design and Town Planning was established in Tirana in 1947, and it was responsible for providing the plans and standard designs for the whole country. These design documents were later modified and harmonized in limited elements to local conditions by local branches. About 300,000 urban and rural flats were constructed during 1945-1979 (Aliaj, 1999).

On the other hand, design and construction practices were centralized in the former Soviet Union, therefore 80% of all Central Asian residential buildings can be categorized as one of only six structural types (King, Khalturin & Tucker, 1996). Most of these structural types were the main template for structural design and construction in Albania as well, during the Communist Regime. Their essential similarities from concept to design and construction methods are presented in the forthcoming sections. These main structural types and the average level of damage expected for different levels of earthquake shaking are described briefly in Table 2.1.

Table 2.1 Central Asian structural types and the expected damage levels (King et al., 1996)

Structural Type	Damage level		
	<i>MSK VII</i>	<i>MSK VIII</i>	<i>MSK IX</i>
1) Non-engineered structures, including small adobe and unreinforced masonry buildings	Heavy damage	Partial to total collapse	Total collapse
2) Brick, bearing-wall systems with wooden floors, one to two stories, pre-19 55	Moderate to heavy damage	Partial collapse	Total collapse

Table 2.1 continues

3) Brick, bearing-wall systems with precast reinforced concrete floors, three to five stories, pre-1957	Slight to moderate damage	Heavy damage to partial collapse	Partial collapse
4) Brick, bearing-wall systems with precast reinforced concrete floors, some seismic detailing, post-1957	No damage to slight damage	Moderate to heavy damage	Heavy damage to partial collapse
5) Precast reinforced concrete frames with welded joints and brick infill walls, four to nine stories	Slight damage	Moderate to heavy damage	Heavy damage to partial collapse
6) Precast reinforced concrete large-panel systems with dry or wet joints	No damage to slight damage	Slight to moderate damage	Moderate damage

In post-Soviet Central Asian Republics, the typical construction method was production of precast reinforced concrete frames, which were composed of linear elements. The reinforcement connection was generally realized by welding. The low seismic resistance of these buildings was due to the location of joints in the areas of greatest load, and the low load-bearing capacity of elements (King et al., 1996). Armenia was hit in 1988 by a large earthquake of magnitude 7 causing more than 25,000 deaths. There were two typical types of structures that were massively destroyed, namely series “1- 451, 1A-450” (4-5 story) and series “111” (5, 9, 12 story) frame panel buildings. The most noted construction flaw in these structures was the eccentricity between the weld spliced vertical reinforcing. The eccentricities in these bars combined with weak concrete in the splice region led to bar buckling and a failure of columns leading to structural collapse (Wyllie, 1992).

Seria IIS-04–1973 was a precast frame type of structure in Uzbekistan, located in zones of high seismic intensity, specifically VII-IX. A technical report from EERI World Housing Encyclopedia (Khakimov & Nurtaev, 2002) described some deficiencies of these structures that may affect their seismic performance. The main problems in these systems are considered the welded joints, due to their brittle behavior in previous seismic events. Joints located in columns or in the beam-column region showed the highest damage. In some cases, non-bearing walls and exterior wall panels have collapsed.

Kyrgyzstan started to implement precast reinforced concrete frames around 1975, and the typical design type was “*series 106*”. The load path of the structure is sustained by the RC space frame and precast floor slabs. So far, no data is present for their seismic performance in major earthquakes. These buildings are not considered to be highly vulnerable to earthquake effects, provided that the construction quality, particularly with reference to the joints, is satisfactory (Begaliev, Uranova & Manukovskiy, 2011).

Masonry structures in Albania built prior to 1990 have been extensively studied (Babalisku, 2014; Guri et al., 2015; Guri, 2016; Korini & Bilgin, 2013; Paci & Bidaj, 2015), hence in this work “Type 5. Precast reinforced concrete frames with welded joints”, designed for the highest seismic zones are the main target for a detailed analysis. These structures were designed to withstand seismic intensities of VIII/IX, however heavy damage and even partial collapse can be the damage level expected for them. Therefore, these structures pose urgent need to be evaluated for their seismic performance and an efficient strengthening scheme should be generated.

2.2 RC Precast Structures in Albania

Some general structural data about old reinforced concrete precast systems in Albania, are provided in the book (Konstruksione Ndërtimi 2, 1987). In absence of other records for the number, type and distribution of these precast structures, reference is taken from this academic source in this section. In the 1980s, the precast reinforced concrete elements were widely used, as they increased the construction speed, efficiency and saved the building materials. Approximately 90% of the residential building slabs and 60% of the industrial building slabs of this time were constructed from precast elements.

In order to understand why the economy of that time in Albania, gave priority to the precast type of construction, a comparison is presented between the precast and cast in place elements. In precast construction:

- Higher grades of concrete could be produced, resulting in subsequent decrease of the concrete and steel quantity demand;
- A shorter construction time was possible and moreover construction work could be performed even in unfavorable weather conditions;
- Less labor was required from workers as production was done in series leading to improved work quality;
- Pre-stressing technique could be used as well with these elements;
- Approximately 80% of the wood material could be saved;

However, there were some other drawbacks for this precast construction technique, such as:

- These elements were produced in full scale and their heavy weight required special equipment for lifting and transportation;
- As these elements needed to be lifted in upper stories, because of the transportation extra reinforcement was required;
- Work execution required great accuracy.

2.3 Precast Slabs

Failure of RC structures during seismic events was generally attributed to the collapse mechanisms of frames, however the performance of slabs has the same importance. Failure of precast concrete buildings in 1964 Alaska, 1976 Tangshan, China, 1988 Armenia, 1994 Northridge, 2001 Bhuj and 2008 Wenchuan earthquakes was mainly due to collapse of floors. One of the main reasons for their collapse were loss of seat due to failure of support system, poor connections, excessive deformation of support system (beam elongation) and deformation incompatibility between the support and floor (Khare, 2011).

Precast slabs were widely applied in Albanian buildings of 1980s. They were classified as follows:

- Precast slabs with ribs filled with RC plates or light-weight hollow blocks;
- Slabs with small/ large panels.

2.3.1 Ribbed Precast Slabs

These slabs were composed of precast reinforced concrete beams (ribs), which were equally spaced and filled in between with RC plates, light-weight hollow blocks, bricks, etc. The distance between these precast beams depended on the type of filling chosen. They were reinforced with normal reinforcement Figure 2.1.a, welded reinforcement Figure 2.1.b or pre-stressed reinforcement Figure 2.1.c.

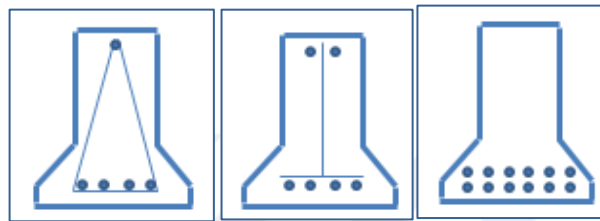


Figure 2.1 (a) (b) (c) Ribbed - precast slabs (Konstruksione Ndërtimi 2, 1987)

The type of a slab with RC plates filling between the ribs is shown in Figure 2.2a. The upper plate formed the floor, whereas the lower plate was supported on the flanges of the beam, forming the ceiling. Between these plates an empty space was created, which provided the slab with high isolating properties. As previously mentioned, the filling between beams was done also with lightweight concrete hollow blocks supported in the beam flanges as shown in Figure 2.2b. The precast slab was alternatively constructed by filling the space between beams with arched bricks. Slabs with span 3-6m were implemented in residential buildings using this technique.

It was required at least 12cm support depth of the beams in the walls, when the beam height varied from 19-25 cm, and 25cm when its height varied from 25-32cm. Every 2-3m the beams had to be supported in the wall. A connection was created with the anti-seismic belt, through the bars coming out of the beam. This connection detail is provided later in this section, and is an important feature to obtain the composite behavior.

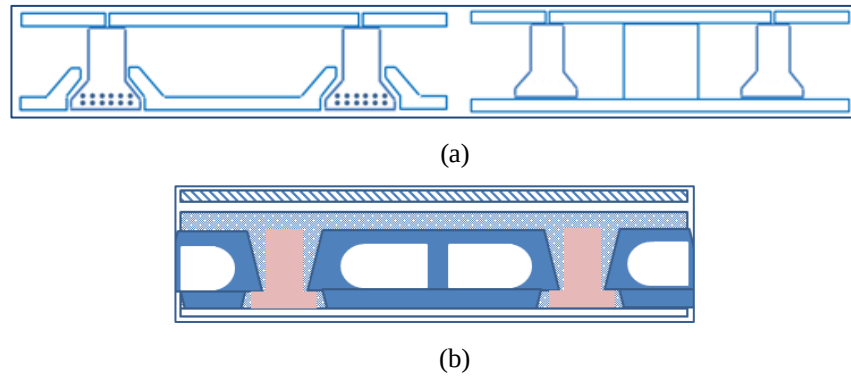


Figure 2.2 Slab with (a) RC plates (b) lightweight hollow blocks (Konstruksione Ndërtimi 2, 1987)

2.3.2 Precast Panels Slabs

In the second type of slabs, the precast elements were placed close to each other. The panel shape and dimensions varied depending on the lifting capacity of the used mechanisms, the concrete grade, the production technology, span, actions applied on it, etc. In Figure 2.3 are shown some of the element types that were widely used in the Albanian construction technology for spans up to 6m.

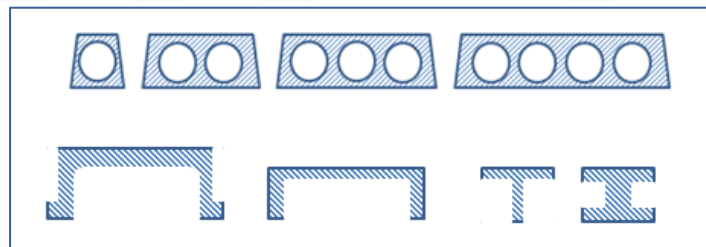


Figure 2.3 Slab with precast panels (Konstruksione Ndërtimi 2, 1987)

Concrete grade M20 and C3 steel class were used. Slabs were formed by placing the panels close to each other and filling with cement grout the empty spaces. In the cases when voids remained between the panels and the walls, they were filled with grout, by placing in the lower part 2-3 steel bars depending on the width of the filling Figure 2.4. In buildings where precast elements were used, in every floor were placed reinforced concrete seismic belts that were always in a lower level than the precast panels.

When creating these slabs, the most important factor taken into consideration was the composite behavior of all elements of the system when subjected to a certain loading regime. This behavior was believed to be obtained by filling with cement grout the voids between adjacent panels.

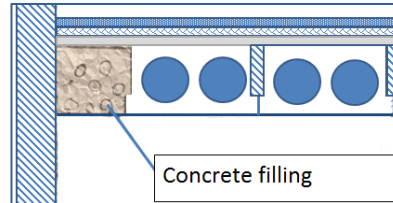


Figure 2.4 Precast slab connection detail (Konstruksione Ndërtimi 2, 1987)

In Figure 2.5 (a,b) is shown the longitudinal-section of a hollow panel, for a live load of 150 daN/m^2 , for span length of $l_0=4.15\text{m}$ (with 2 and 3 hollow cores). The upper longitudinal reinforcement serves as well for the connection of the panel with anti-seismic belt.

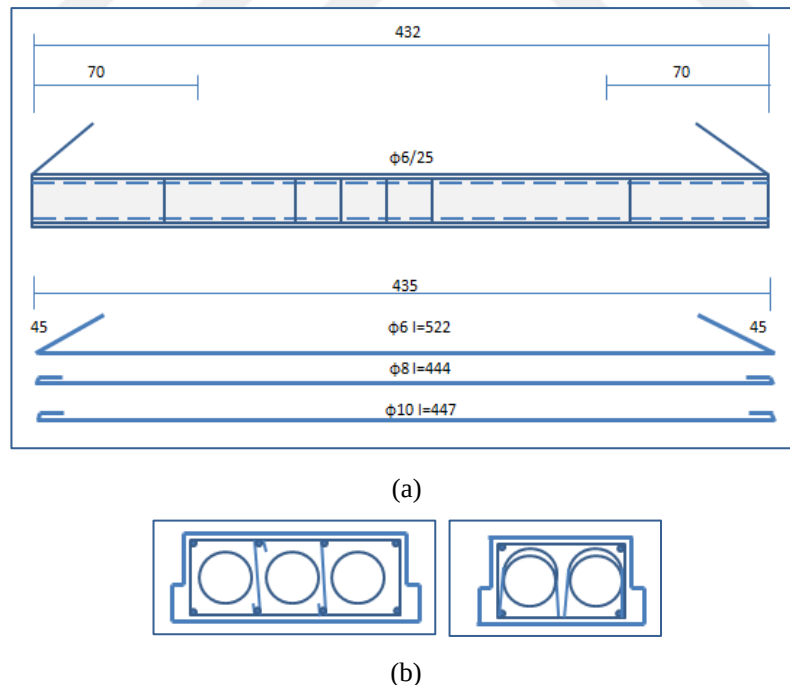


Figure 2.5 Precast panel (a) longitudinal- section (b) cross- section (Konstruksione Ndërtimi 2, 1987)

2.4 Connection of Precast Elements

In precast constructions, the most important element is the overall stability and rigidity of the structure in space. For the panels, it is very important to attain the necessary support on the wall. Practice has shown that the support depth of the panels, depending on the element height, the wall width and the region seismicity, should fulfill the following:

- The support depth of panels with height $h \leq 15\text{cm}$, in the outer walls with thickness $t=25\text{cm}$, should be 10cm.
- For panels of height more than 15cm, the support depth in the outer walls of thickness $t=25\text{cm}$ is taken 12cm, for seismic intensity VII and 15 cm for seismic intensity VIII and IX.
- In the inner walls the support depth, is independent on the seismicity of the construction zone and the height is taken 10cm.
- When the wall has a thickness of 38cm, the support depth can be taken with a larger value.

In order to create the connection, in the anti-seismic belt every 50cm are placed steel bars of $\phi 10\text{mm}$ diameter. The bars that come out of the panels are hooped in the end like shown in Figure 2.6. Two longitudinal steel bars of $\phi 10\text{mm}$ are placed and then carefully cement grouted.

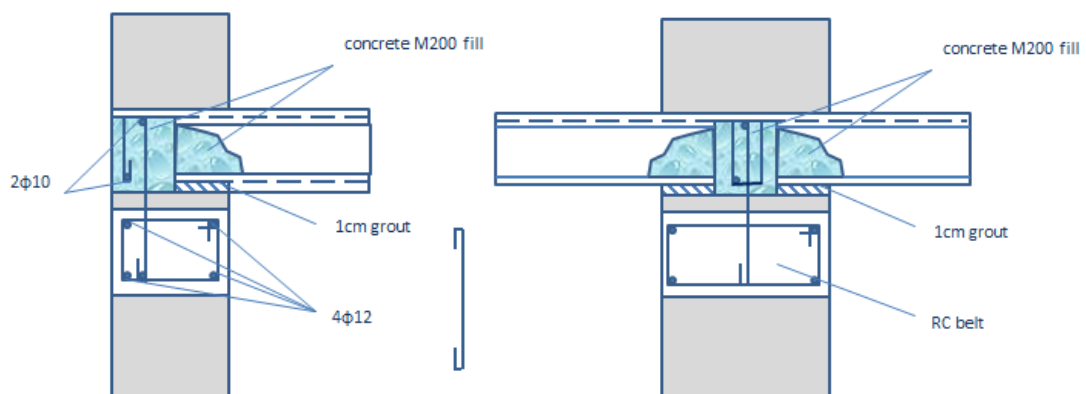


Figure 2.6 Precast connection detail (slab and seismic belt) (Konstruksione Ndërtimi 2, 1987)

2.5 Precast Beams

A representation of the most common precast beam types used in Albania is provided in this section. A historical devastating earthquake can be recalled here in order to stress the importance of the seismic behavior of beams in precast construction. On December 7, 1988, an earthquake of magnitude 7.0 hit the northern region of Armenia causing more than 25,000 deaths. One of the primary reasons for the severe damage was the frame weak link: beams in shear. Extensive shear cracking was observed in the beams at the end of the steel angle embedded in the column (Wyllie, 1992).

Precast reinforced concrete beams in Albania were classified as: simple one span beam, beam with “gerber” system and continuous beams Figure 2.7. In Albania, the first and second group of beams were used mostly. Continuous beams were generally applied for industrial buildings, as their monolithic joint in the support was quite easy to obtain. These beams had different cross sections such as rectangular, T-section, I-section, II-section, etc. The section type was chosen based on the building function. Precast beams were constructed with concrete of grade M300 and cement of grade M50. The beams were reinforced with steel bars of grade C3 with $R_a=210\text{MPa}$.

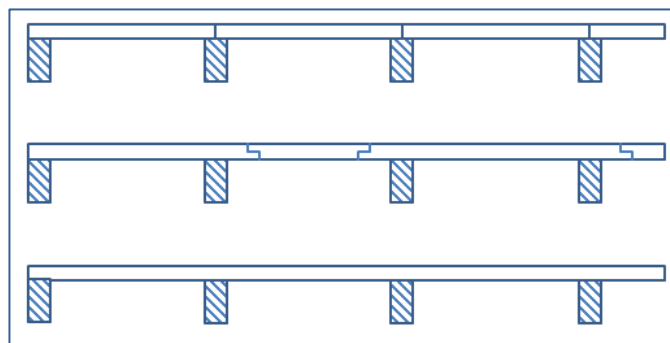


Figure 2.7 Precast continuous beam (Konstruksione Ndërtimi 2, 1987)

Beams were cast in two ways, namely in vertical and horizontal positions. The connection of the beam with the columns was generally realized by welding the metallic plates of both elements to each other as shown in Figure 2.8.

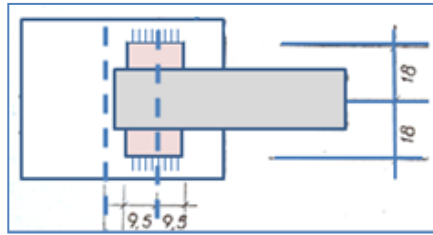


Figure 2.8 Beam-column connection with welded metallic plates (Konstruksione Ndërtimi 2, 1987)

In the beams of the “gerber” system, the connection was realized by just using a steel hook (stud) that simulated the behavior of a free support as shown in Figure 2.9. Their disadvantage is that their application increases the construction cost. These systems were generally preferred to minimize the residual deformations.

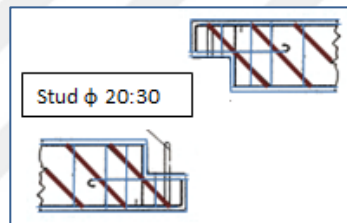


Figure 2.9 Connection in “gerber” system beams (Konstruksione Ndërtimi 2, 1987)

Continuous beams were preferred in the cases when dynamic or seismic actions were present in the structure. Attention was paid to avoid their application in the systems prone to differential settlements. In this type of beams, the joint construction was generally complex to realize, by crossing the longitudinal bars along the joint over the support and filling with concrete thereafter (Figure 2.10).

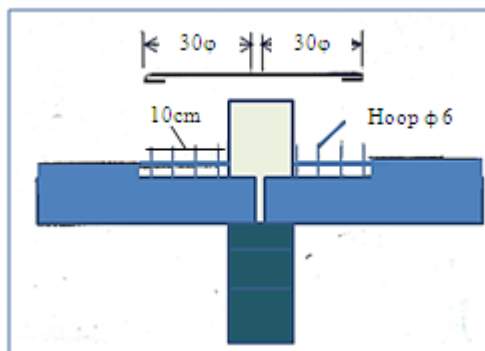


Figure 2.10 Joint type in continuous beam (Konstruksione Ndërtimi 2, 1987)

2.6 Precast Columns and Foundations

In this section a representation of the most common precast column types used in Albania is provided. Columns are the main load carrying elements in most of the precast systems in this country. Failure of these types of elements in the past has generated total collapse of structures, such as during Armenian earthquake in 1988. Column splices with eccentricities between the weld spliced column vertical reinforcing bars combined with weak concrete in the splice region led to bar buckling, failure of columns and finally total collapse (Wyllie, 1992).

The columns shape depends on the structural working scheme. In Albania, in one-story buildings these columns are cast in one full story height. Whereas in multistory structures these columns are cast either full-height or they are divided and then later welded into one full height column. Their cross section is generally rectangular or I shape. The columns with flanges per each floor are generally used when large spans are needed and when the equipment lifting capacity is more than 10t. Reinforced concrete precast columns are calculated and constructed in the same way as the monolithically cast reinforced concrete columns. In Figure 2.11 are shown the construction details of a precast reinforced concrete column.

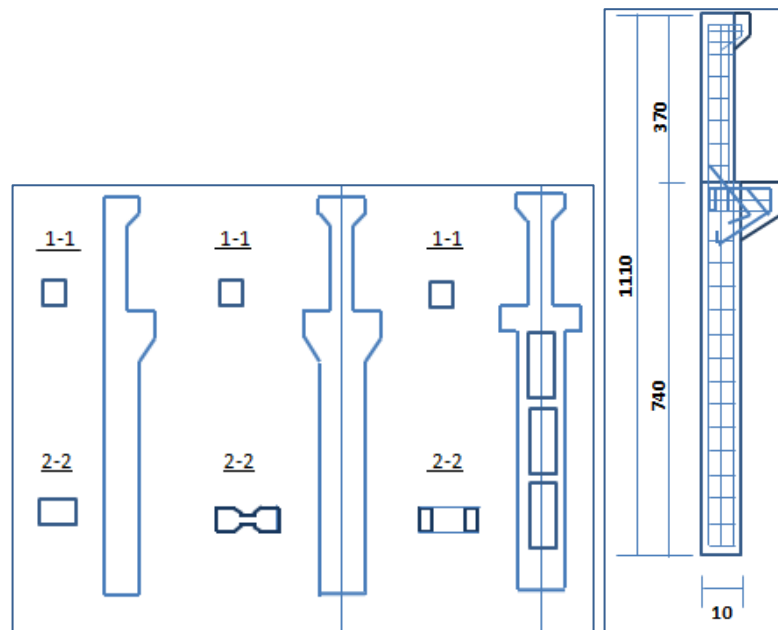


Figure 2.11 Precast RC column types (Konstruksione Ndërtimi 2, 1987)

These precast columns are placed in the respective foundation. Mostly used are the reinforced concrete monolithic pad foundations, which are cast in place by leaving a gap open for the column placing, as shown in Figure 2.12.

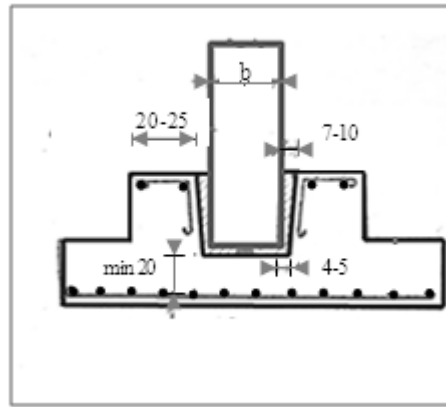


Figure 2.12 Socket type pad foundation (Konstruksione Ndërtimi 2, 1987)

Once the column is placed in the correct position, it is fixed on the four sides. After the central positioning, the spaces between the column and the foundation are filled with concrete. The spaces between the column and foundation should have the following range: 4-5cm in the lower part, 7-10cm in the upper part. The gap depth shouldn't be less than the larger cross section dimension of the precast column. On the other hand, the thickness of the bottom part of the foundation shouldn't be less than 20cm.

CHAPTER THREE

TECHNICAL DETAILS OF A PRECAST FRAME STRUCTURE

3.1 General Information

The structure type selected for analysis in the forthcoming section is a precast reinforced concrete frame structure. This structure was designed in 1982 for the highest seismic zones in Albania. The full project design details were obtained from the archives. The building types shown in this project, identified namely as 82/1 and 82/2 were planned for high seismic areas. In this section only some representative cases of the structural elements are presented. This structure was chosen to be evaluated whether its performance is satisfactory when subjected to a design level earthquake, because:

- It was designed for the highest seismic zone;
- The seismic hazard map used during design is not compatible to nowadays standards;
- The material properties used in construction and detailing requirements may not conform to the latest seismic requirements.

3.2 Structural Details

The pad foundations, as shown in Figure 3.1, are designed for the allowable bearing capacity of 15kg/cm^2 and 18kg/cm^2 . They are placed at the same level, but not less than 1.5m from the surface. In case soil cracks are present, they have to be placed minimally 40cm deeper than these cracks. In case bad soil conditions are present, these pad foundations need to be re-designed with higher necks in order not to change the column total length. The pad foundations are constructed with concrete of grade M-150, and laid above a 10cm layer of M-100 concrete. The pads are interconnected in the top with precast foundation beams. They are placed in the perimeter, as well as in the inner area, respecting a limiting space of maximum 20m.

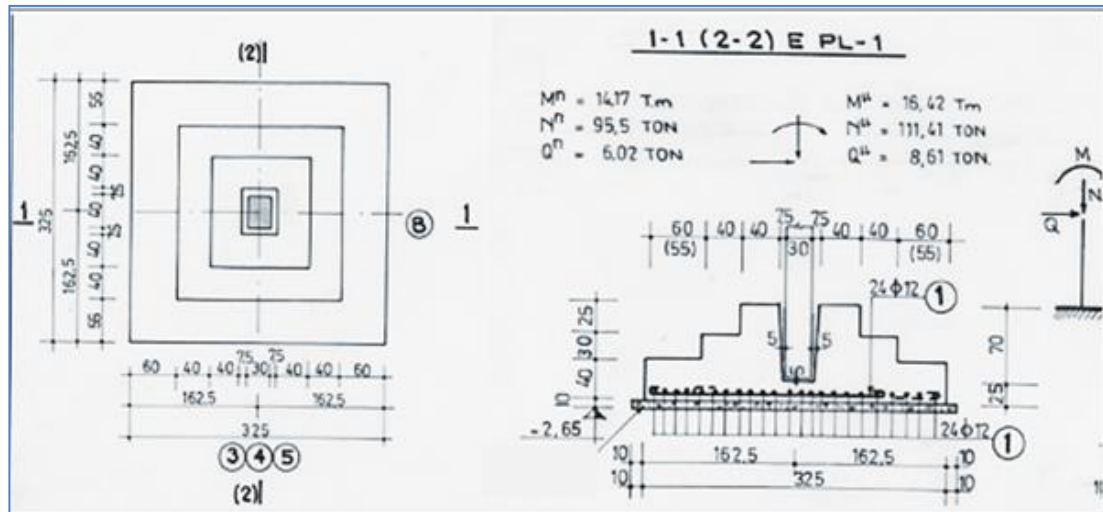


Figure 3.1 Detail of pad foundation type (Banesa tip, 1982)

Despite the pad foundation construction details, in the project drawings are included also the ultimate axial force, shear force and moment that they were designed to resist. This is a very helpful data, once the strengthening is performed so that it can be deduced whether further intervention in the foundation is needed. In order to create a horizontal diaphragm, between the precast slabs $\phi 8$ mm bars are placed, as shown in the project. The monolithic joint between the beam and the slabs is realized based on the detail given in the Figure 3.2, and the voids are filled with concrete.

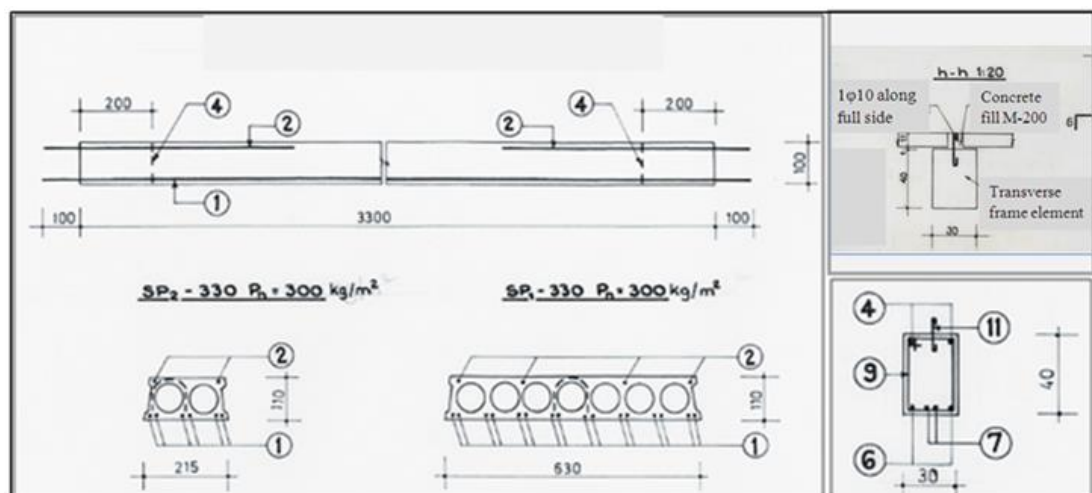


Figure 3.2 Precast slabs and detail of beam connection to precast slab (Banesa tip, 1982)

Whereas the monolithic joint located within the beam is realized as shown in Figure 3.3. The precast elements of the frame are produced with concrete of grade M-300 and steel $R_{ak}=210\text{kg/cm}^2$. In the detail of monolitization of the beam and column joint, the surface should have the texture as shown in the project, in order to create a better bonding between the old and new concrete.

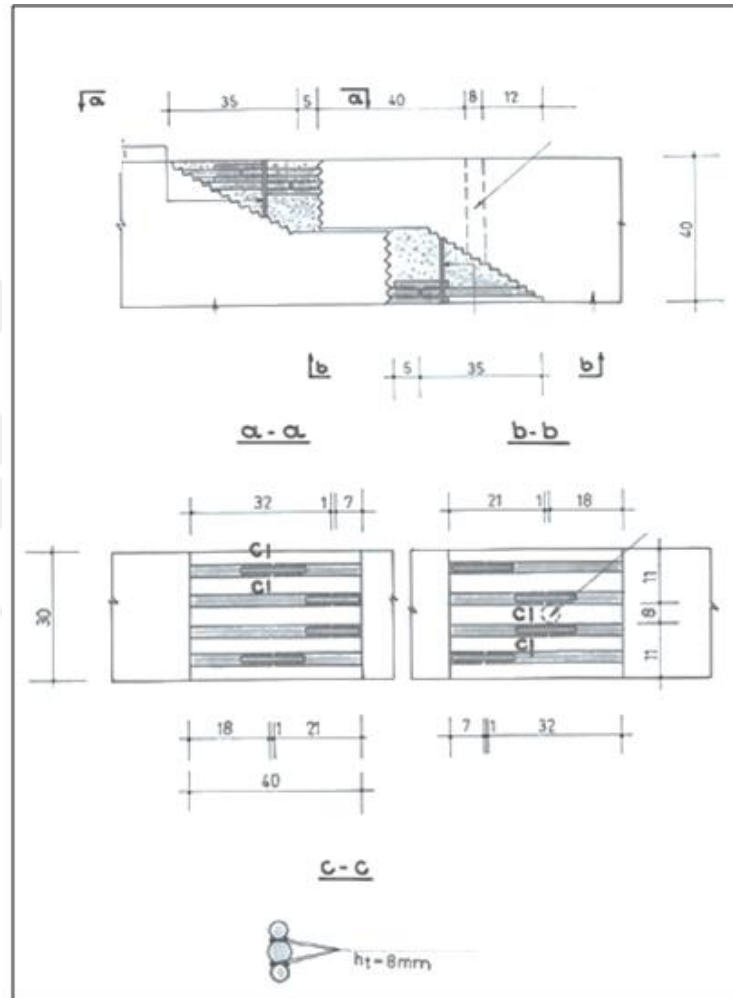


Figure 3.3 Detail of beam-column joint (Banesa tip, 1982)

Columns are placed in the pad foundation necks as shown in Figure 3.4 and they are filled with concrete of grade M-300. After the concrete has reached 70% of its strength then the upper elements placement starts. Simulated cast-in-place joints have to be obtained in all joints of the superstructure frame elements. **Construction instructions emphasized the importance of realizing the welding without making an axis deviation.**

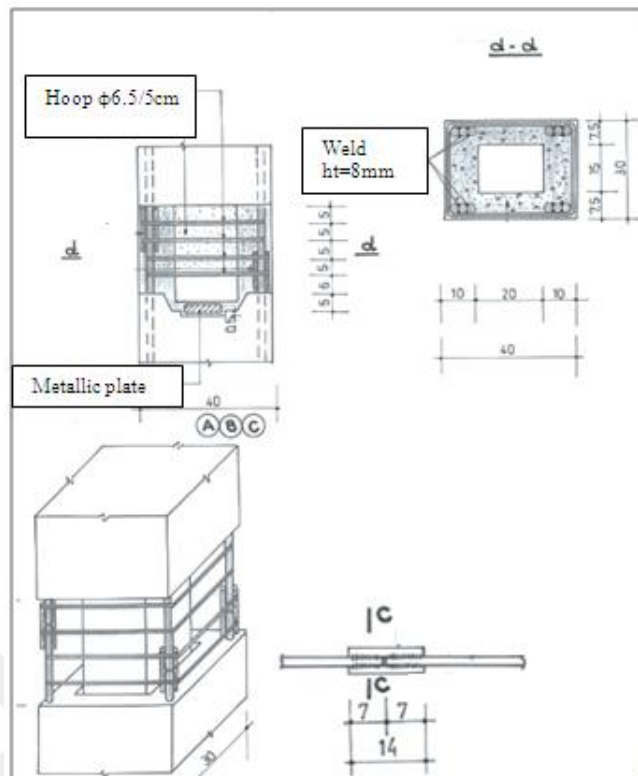


Figure 3.4 Column-column connection (Banesa tip, 1982)

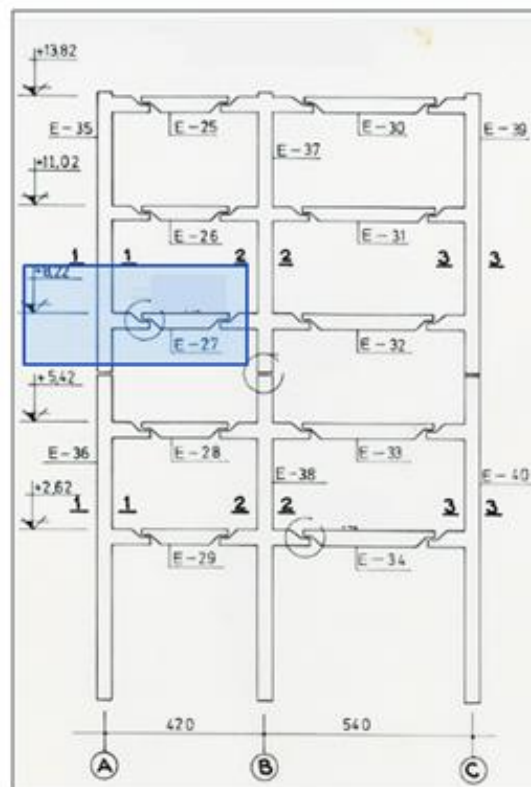


Figure 3.5 Frame in transverse direction (Banesa tip, 1982)

The structure was composed of two-bay frames repeated in the longitudinal direction, up to five spans. The frame shown in the Figure 3.5, is the one representing the most flexible direction of the structure. It has to be recalled here that the project represents a type-model structure, and the number of spans in the longitudinal direction are not fixed, but they vary from minimum 5 spans. Hence this frame is designed for several structures, with 5 or more spans in the longitudinal direction. As a result, the strengthening of this specific frame is considered in this section.

As additional construction details, it can be mentioned here that before concrete was added, the steel cleaning from corrosion and wetting of the concrete was performed in order to obtain a better bonding. The inner and outer walls are composed of hollow bricks 20x22x12cm and 8x11x23cm and with M-15 grout.

3.2.1 Precast Slab Behavior

Precast slabs are some structural elements that were widely applied in Albanian buildings of 1980s. In the current case study, hollow core precast slab panels are used, which are placed close to each other and grout filled. Their monolithic detail is shown in Figure 3.6.

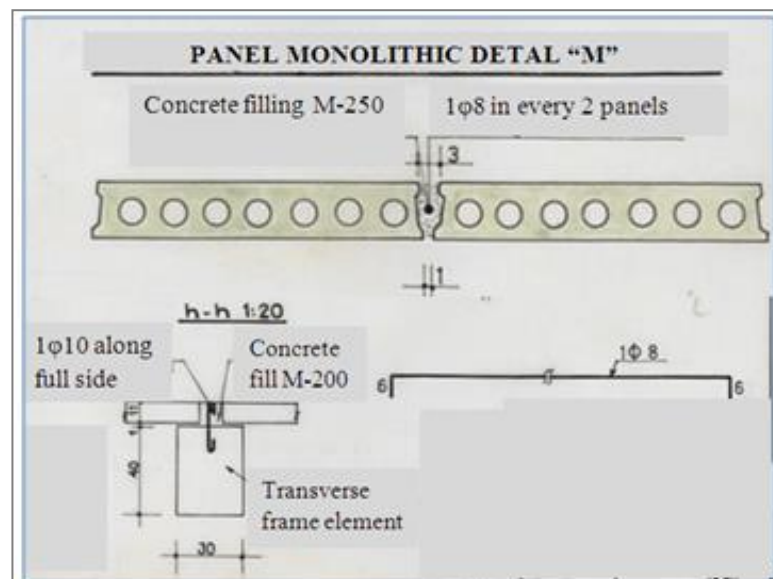


Figure 3.6 Details of monolithic joint in precast hollow core panels (Banesa tip, 1982)

The behavior of the precast panels in the structures subjected to Armenia earthquake (Wyllie, 1992) is recalled in this section. The main problem in these units was lack of proper interconnection. Grout was present between individual units, but no reinforced topping slab, no connection to adjacent units was present, with only the beam reinforcement at each column line crossing the joint plane.

The earthquake consequences made clear all the design and construction flaws. There were significant problems in the concept and construction of proper connections between the columns and the slabs. The extreme differences in rigidity between floors and columns, adding to the weak diaphragm created by the slab itself, prevented the proper load distributions once the very first failure occurred in the structure (King et al., 1996).



Figure 3.7 Floor plank failure (Clinton, 2009)

Figure 3.7 shows the failure of floor planks of a building under construction. The hollow core floor planks were only simply supported on the beams over a 5cm to 8cm seat. Interconnection between units was absent as no ties between the planks or between the planks and the beam reinforcement were observed. Whereas in Albania, in buildings where precast elements were used, in every floor were placed reinforced concrete seismic belts that were always in a lower level than the precast panels.

When creating these slabs, the most important factor taken into consideration was the composite behavior of all elements of the system when subjected to a certain loading regime. This was done by filling with cement grout the voids between adjacent panels. In order to create the connection, in the anti-seismic belt every 50cm were placed steel bars of $\phi 10\text{mm}$ diameter. The bars that came out of the panels, were hooped in the end like shown in Figure 3.8. Two longitudinal steel bars of $\phi 10\text{mm}$ were placed and then carefully cement grouted.

This configuration has many similarities to the precast reinforced concrete frame buildings (series 106) that were introduced in Kyrgyzstan around 1975, in which reinforcement bars are projected from the slabs for achieving the anchorage to the beams. The difference is that the seismic belt and the precast unit are not in the same level as in the 106 series structures. Buildings of this type have not yet been subjected to major earthquakes. These buildings are not considered to be highly vulnerable to earthquake effects, provided that the construction quality, particularly with reference to the joints, is satisfactory. In Series 106 structures, the roof diaphragm is considered to be rigid and it is expected that the roof structure will maintain its integrity, i.e. shape and form, during an earthquake of intensity expected in this area (Begaliev, Uranova & Manukovskiy, 2011). The similarity in design and construction of this type of structures to the Albanian types, may suggest that these slabs can maintain their integrity during a seismic event. However, further experimental research is needed to verify their real performance.

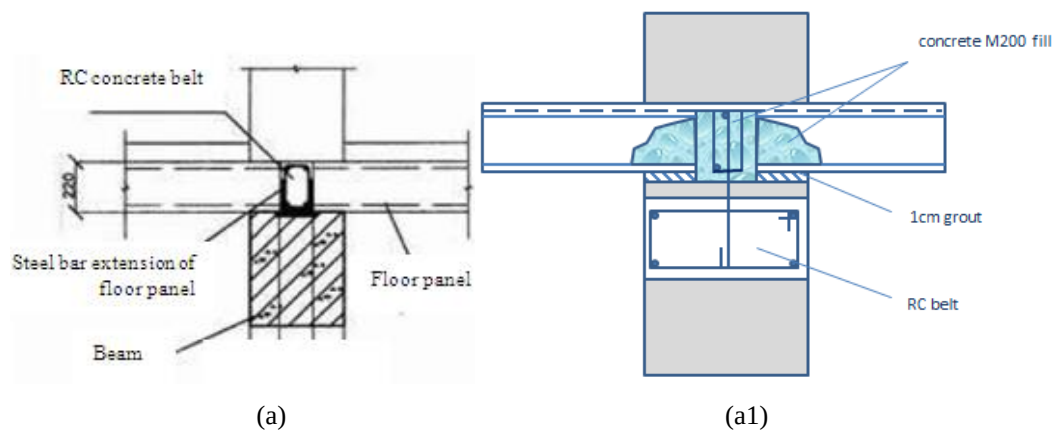


Figure 3.8 Precast connection detail (a, b, c – Begaliev et al., 2011) and (a1, b1, c1 – Albania)

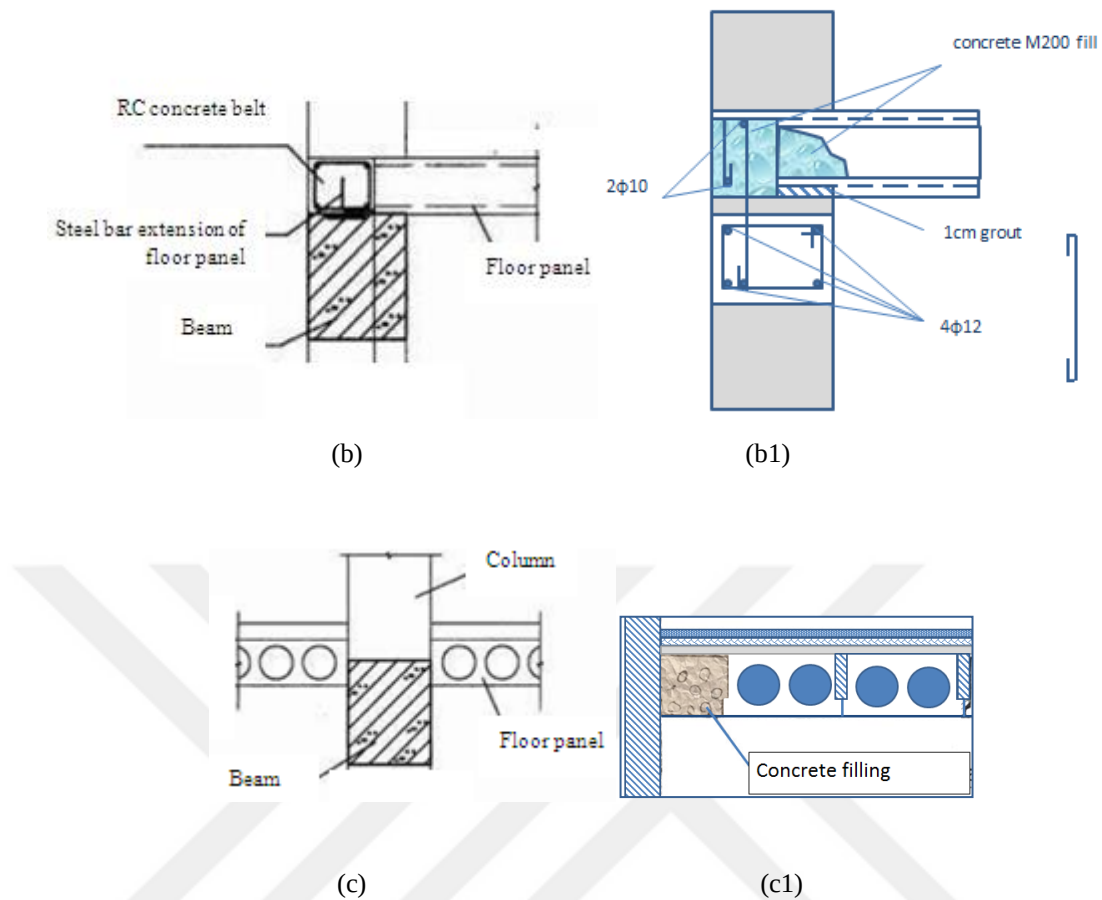


Figure 3.8 continues

3.3 Precast Joint Types

Precast construction can produce different types of joints between structural elements, from pinned to simulated - monolithic joints. Before stepping into the necessary joint analysis of the structure that is currently under review, in this section the seismic performance of some similar joints that have been subjected to significant earthquakes is provided. Seismic performance of Series 111 structures gave some important lessons about the behavior of welded joints. The crucial problem observed was the wrong location of these welded joints. In the cases when their position corresponded to the zones showing highest tension, they were not able to undergo ductile deformations or have any reserve load bearing capacity. Welded joints had poor construction quality and an insufficient number of reinforcing bars, resulting in 23% lower strength than required (King et al., 1996).

In these precast frame structures (Wyllie, 1992), the precast columns were cast in two story lengths and spliced near mid-height as is the case for Albanian type. The design details in the respective project showed that four of the six bars of the column had to be aligned and butt welded with full penetration welds. However, during inspection on site after the seismic event, these details were never present. Instead of the butt welding with full penetration welds, a third shorter piece of reinforcing steel matching in diameter with the longitudinal reinforcement was side welded to the bars in each joint plane. This wrong procedure created an eccentricity between the longitudinal column reinforcement. Moreover, the weak concrete present in these joints combined with aforementioned eccentricity became the main factors that led to bar buckling and failure of columns. This scenario led to overall collapse of the structures.

A technical report from EERI World Housing Encyclopedia (Khakimov & Nurtaev, 2002) describing the design properties of the “Seria IIS-04 - 1973” type of structure in Uzbekistan, deduced that the main problem was considered the welded joints, as they have shown extremely brittle behavior during seismic events. These buildings showed considerable damages during the 1988 earthquake, as illustrated in Figure 3.9. A schematic representation of the structural details and the principal construction problems are presented in Figure 3.10. On the other hand, construction instructions in Albania emphasized the importance of realizing the welding without making an axis deviation. Therefore, in this study the eccentricity in welding column longitudinal reinforcement is not taken into consideration.



Figure 3.9 Building damage 1988- Armenia Earthquake (Clinton, 2009)

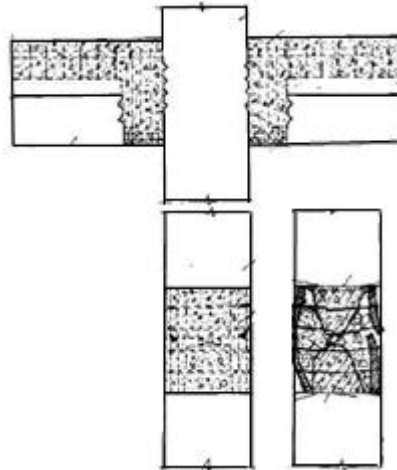


Figure 3.10 Critical seismic structural details and deficiencies (Khakimov & Nurtaev, 2002)

In the structure currently under review (Appendix E), the designers have highlighted that they wanted to obtain joints that simulate the monolithic ones. Hence in this case, before any further modeling is performed, the proper classification of these joints is needed. A typical joint type is shown in Figure 3.11.

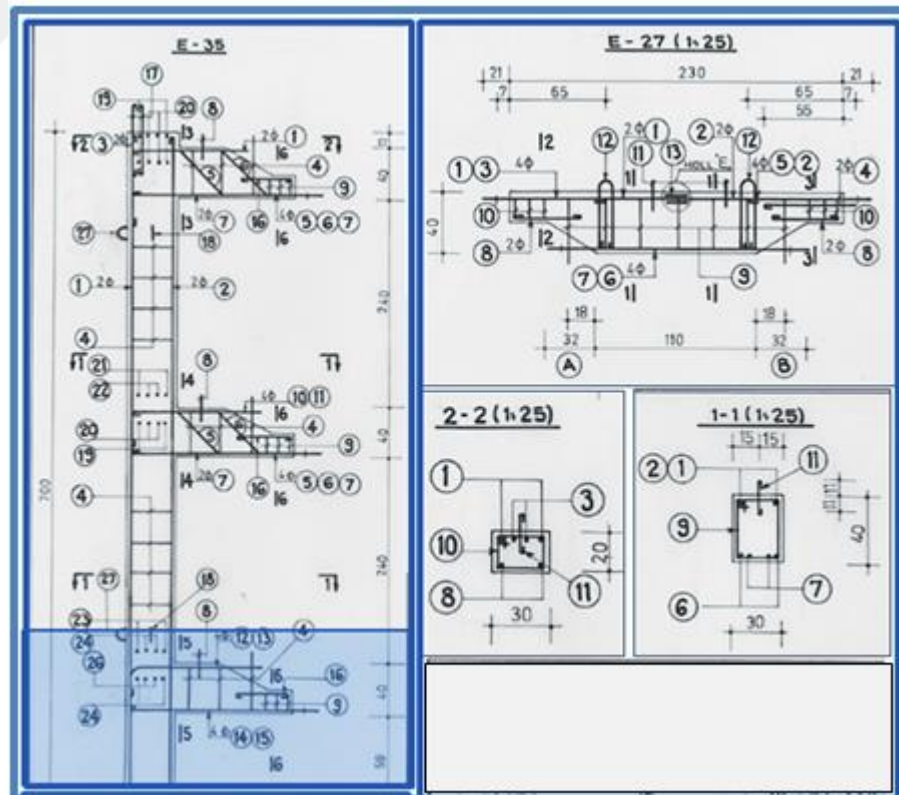


Figure 3.11 Details of beam joint type “N” (Banesa tip, 1982)

As it is seen from the schematic representation of the reinforcement in the structural elements, the top and bottom reinforcement was respectively 4 ϕ 20 (top) and 4 ϕ 14 (bottom). Here are highlighted the two fundamental criteria that determine the plastic hinge location. First of all is recalled here that when the seismic forces are present the maximum moment of the beam occurs at the extremities, hence near the column faces. As the same reinforcement is present in this region (sections 1-1 through 4-4), yielding is expected in the region between the column face and the precast joint where the generated moment will be maximal (Figure 3.12).

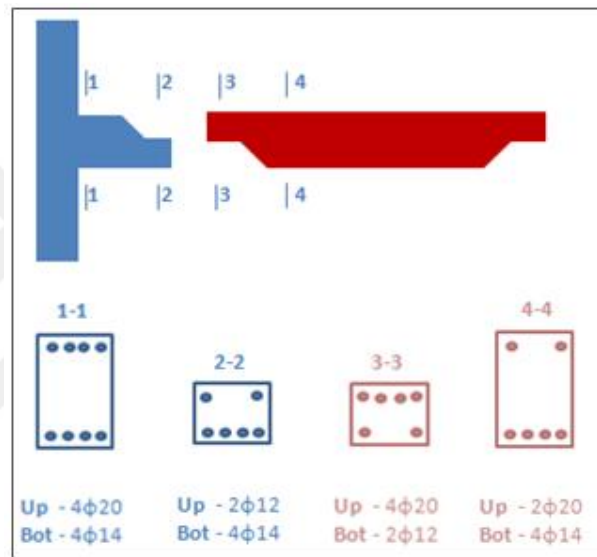


Figure 3.12 Schematic representation of the reinforcement

The next step is the classification of this joint type as ductile or strong, according to the basic (ACI:318-14, 2014) requirements. For moment frames constructed using ductile connections, it is required that plastic hinges are able to form in the connection region. Whereas in strong connections, the coupler type needs to relocate the yielding away from the connector, into the bar itself (ACI:318-14, 2014).

As the experimental results (Scholz & Roberts, 2000) suggest, the “*indirect butt double bar splice*” behaves the same way. It allocates the yielding beyond the connection region; hence it can be considered as a strong connection. In these conditions again the yielding is expected in the region between the column face and

the precast joint, as the deformations in the plastic region of a beam suggest in Figure 3.13 (Fenwick & Dhakal, 2007).

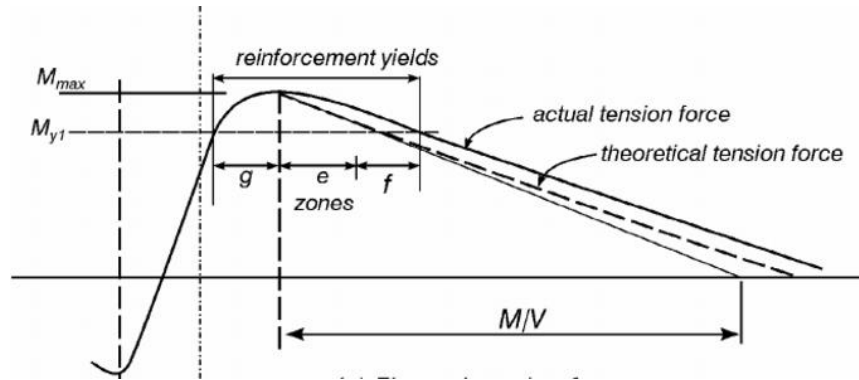


Figure 3.13 Flexural tension force along a beam (Fenwick & Dhakal, 2007)

A sample of a not recommended strong type connection is presented here. In the case when the reinforcement diameter in the beam-column junction is the same, and the strong connection is located just at the face of the column, yielding is expected precisely at the face of the column. Moreover, it will not be able to spread into the beam due to the strength of the connector. In this way the plastic hinge will not be able to develop and the yielding will concentrate to the bar just next the connector on the column face, which will probably fracture due to large rotation imposed on the beam (Nakaki, 2013). Whereas in the current case the strong connection is located in a distance from the column face as much as the beam depth or more, hence this is a favorable case that suggests that yielding can spread in the beam up to the column face and not have concentrated damage.

3.4 Existing Building Deficiencies

The seismic hazard present in Albania and low construction material quality are the main factors that pose the need for the seismic performance evaluation of old structures. However, the design code of the construction time plays as well a major role in the structure performance. In this section is provided in schematic format a review of the frame detailing, compared to nowadays requirements. In Figure 3.14 the details that do not conform to recent standards are noted in red, whereas the ones

that conform are highlighted in green. In this checklist, attention is given to the critical parts that are assumed to dissipate energy in a frame, where yielding is expected. In this perspective it is observed that the main beam reinforcement is continuous up to the column outermost face, and the lap splices in columns do not occur in the most stressed regions.

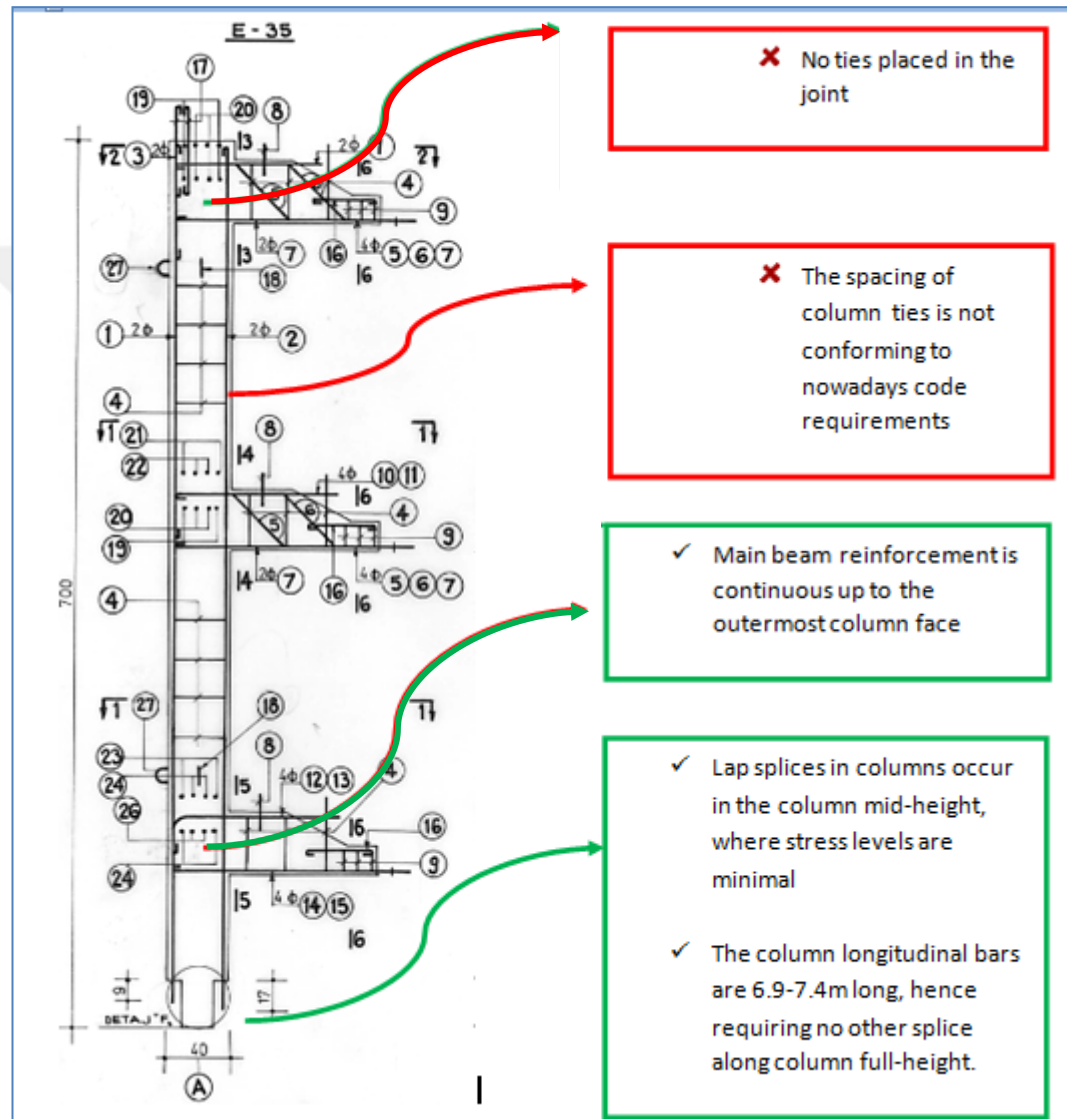


Figure 3.14 Construction details evaluation

These two characteristics were mainly observed as deficiencies in other countries for the old construction type (ATC, 1996), and were some of the main causes for the structure failure. However, the problem observed in these frames, is the confinement

of elements. Neither the transverse reinforcement in the column, nor within the joint meets the recent criteria for seismic design.

On the other hand, an important structural element that needs to be checked is the foundation. It is well known how retrofitting strategies of the structure, impose extra demand in the existing foundation, requiring an intervention which is quite costly. The type of the foundation designed for the precast frame under consideration, is a precast socket type pad foundation. In order to avoid differential settlement, foundation link beams were used as well in the perimeter of the structure, and as well within the inner pads if the distance was significant. The cross section of one pad foundation is presented in Figure 3.15.

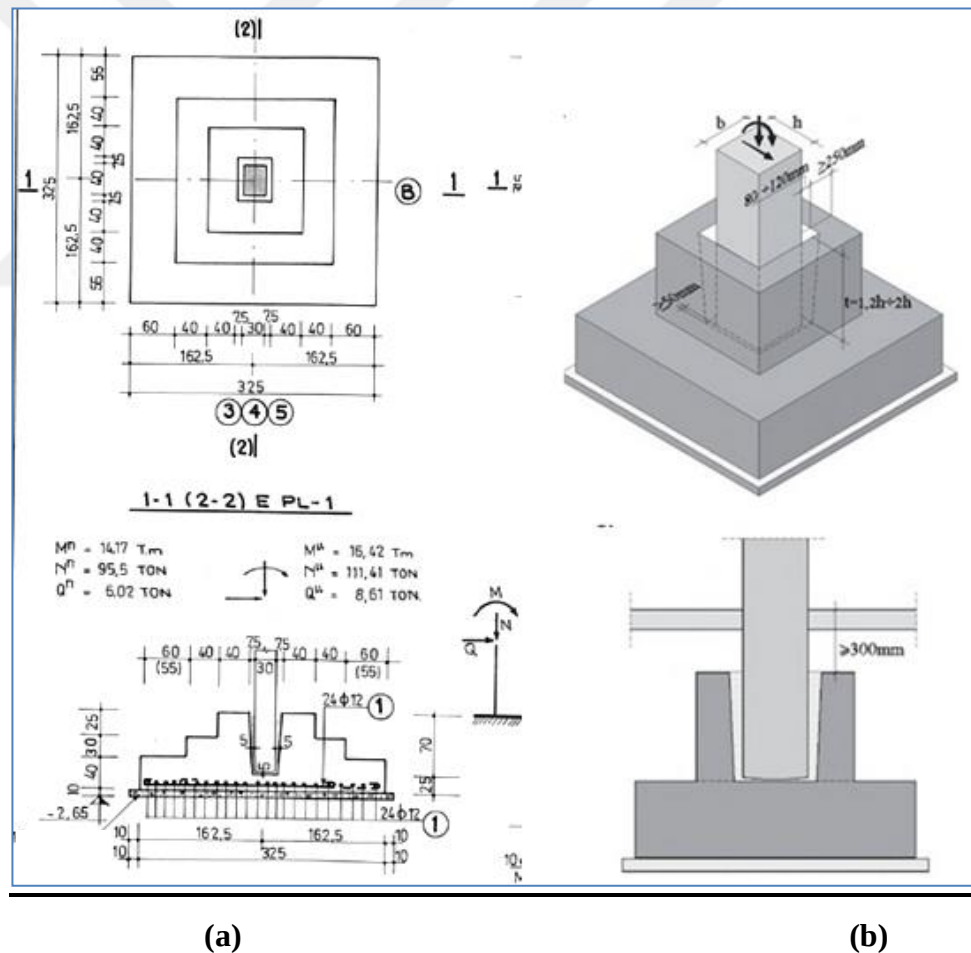


Figure 3.15 (a) Original pad foundation (b) Recommended dimensions (Du Beton, 2016)

Comparison is done referring to the recent studies (Du Beton, 2016), for the recommendations in the socket foundation basic dimensions:

- The distance between the ground surface and the upper face of the socket fulfills the minimum requirement 30cm.
- The socket wall thickness fulfills the minimum requirement 25cm.
- The depth of column in the socket is 70cm, conforming to the required height $1.2h-2h$.
- The distance between the socket and column faces fulfills the recommendations.



CHAPTER FOUR

SEISMIC HAZARD IN ALBANIA

4.1 Seismic Risk

Albania is considered an active seismic region and adequate seismic design in this country should target seismic-safe structures. In this section is presented the seismic map that is actually used as reference for the design of seismic structures in Albania (KTP-N.2-89, 1989) Figure 4.1. This map shows the maximal expected intensities MSK-64 for 100 years return period. Albania's territory is classified in three regions with intensity MSK-64 (VI, VII and VIII). Intensity IX is attributed to some local regions due to soft soil conditions (not shown in the map). It is obvious from this map that a large part of Albania is risked by significant possible seismic events. The scale used in this map originates from the former Soviet Union, where seismic intensity is measured on a 12-step scale, called the Medvedev-Sponheuer-Karnik (MSK) scale.

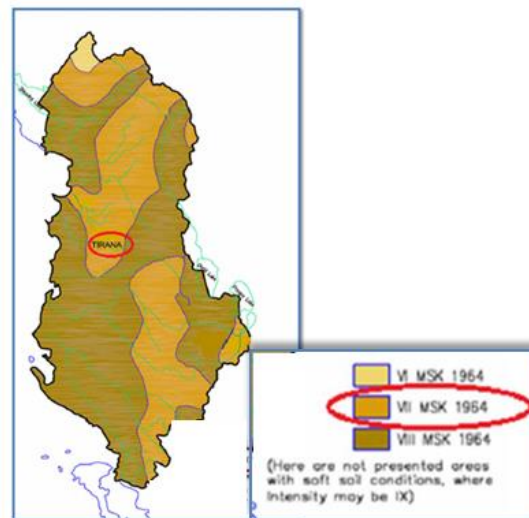


Figure 4.1 Seismic map currently used in the Design Code KTP-2 1989

A portion of this scale is shown in Table 4.1, and is similar to the Modified Mercalli Intensity (MMI) scale used in the United States and Europe.

Table 4.1 Partial definition of MSK intensity scale (King et al., 1996)

MSK intensity	Consequences	
	<i>People</i>	<i>Buildings</i>
VII	Frightened	Poor-quality structures considerably damaged; ordinary structures slightly damaged
VIII	General Fright, some panic, difficulty standing	Poor-quality structures collapsed; ordinary structures considerably damaged; and well-built structures slightly damaged
IX	General panic	Many ordinary structures destroyed; well-built structures heavily damaged
X	Thrown to ground, strong disorientation	Most buildings destroyed, including some well-built structures

4.2 Earthquake Type

Albania and the Balkan peninsula where it is located are known for the considerable seismic activity, as they belong to the Alpin-Mediterranean system (from Azore Islands along the Mediterranean) which extends east along Asia and is joined with the Pacific System in Malaysia. The system itself is responsible for approximately 15% of the total energy released from all the earthquakes all over the world. Because of the complicated tectonics in the Carpates and the multiple interaction of the tectonic in the Aegean and Adriatic seas, the Balkan peninsula is considered the principal seismic region in the continent (Aliaj et al., 2010).

Some typical characteristics of the earthquakes in Albania are summarized here. The earthquakes are generally shallow with depth varying from 10-20km. Moreover, there are cases when these earthquakes may even reach the surface. However, no earthquakes of magnitude larger than 7.2-7.5 are expected as the rocks present in this area are characterized by limited strength. In this scenario it is not possible to accumulate enough strain such as to produce such major events, but in many cases a different type of energy release occurs generating a set of earthquakes of small to medium magnitudes.

4.3 Historical Records

Statistical data show that from the period of III-II centuries B.C up to now 89 strong earthquakes hit Albania. However, these data can probably underestimate the real seismic risk, because approximately 60% of these recorded earthquakes belong to the 19th century. This lack of data from the early past combined with the fact that 20 of the recorded earthquakes have had the intensity of IX degree, leaves these historical records with the big mystery of how many disastrous earthquakes did actually hit our country.

Recently, another strong earthquake of magnitude 6.4 hit Albania on 26 November 2019 (U.S. Geological Survey, 2019). Unfortunately, 51 people died and more than 3000 were injured, as it was the largest earthquake that has hit Albania in the last 40 years. It was not a single major event, but as predicted 1300 aftershocks have occurred until 1 December 2019, 4 of which had a magnitude larger than 5.

The most recent significant earthquake in the area was an M 5.6 event on 21 September 2019 with epicenter in Durrës, which damaged 500 houses as it was registered as the most powerful earthquake in the last three decades (Withnall, 2019). The buildings that totally collapsed were two hotels and two flats in Durres, whereas in Thumane, 4 buildings including a 5-story flat were destroyed.

4.4 Seismoactive Fault Zones

Three longitudinal and two transverse active fault zones are evidenced into the Albanian orogen as follows (Aliaj et al., 2010):

1. The Ionian-Adriatic thrust N-W
2. The Shkodra-Mati-Librazhdi graben N-W
3. The Peshkopi-Korca graben N-S
4. The Shkodra-Tropoja normal N-E
5. The Elbasani-Dibra normal N-E

4.5 Seismic Hazard Assessment – Probabilistic Approach

The seismic hazard evaluation in Albania has evolved considerably in the recent decades, from an empirical system present before 90s up to the most recent trends to produce a probabilistic seismic map. Specifically, before the 90s macroseismic intensities of strong past seismic events and seismotectonic synthesis were the basis of the developed methodology for the seismic hazard evaluation (Sulstarova, Kociaj & Aliaj, 1980). It should be noted here there that these methodologies lacked of an accurate mathematical applicable module, and they can be mostly considered as empirical in nature. However, there has been considerable research in the recent years attempting to provide both a deterministic and probabilistic methodology to relate the seismic hazard itself with ground acceleration, velocity and displacement (Duni & Kuka, 2003; Duni & Kuka, 2004; Fundo, Duni & Kuka, 2012; Muco, Vaccari & Panza, 2001; Muco et al., 2002).

A recent study (Baballëku, 2014) has presented a direct comparison between elastic spectra produced from different seismic maps in time, where it is clear that actual Albanian code based seismic parameters need to be improved, in order to provide a proper seismic hazard design map for the country. In Figure 4.2 are presented the elastic spectra produced referring to the seismic provisions of KTP-N.2-78 (acc. to seismic map '63), KTP-N.2-89 (acc. to seismic map '78) and EN 1998-1-2004 (acc. to seismic map '04).

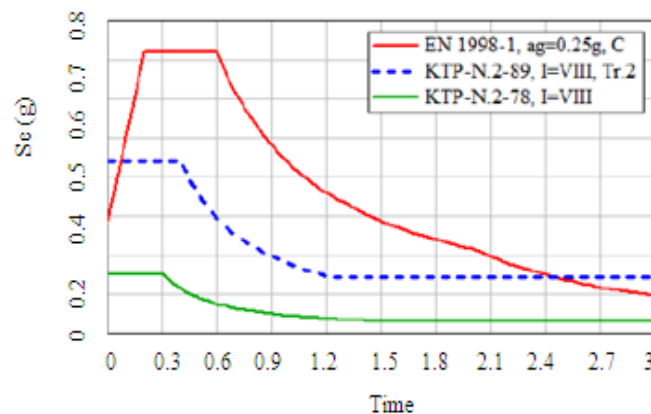


Figure 4.2 Comparison of elastic spectra based on seismic provisions (Baballëku, 2014)

Hence in this study a probabilistic approach is used to generate seismic hazard of the region under consideration. The probabilistic approach in simple terms can be explained as how possible it is to observe different magnitudes of earthquakes at a site, which when converted in mathematical terms defines the annual probability that a certain magnitude of seismic event is expected to be exceeded in the specific area under consideration. The first assessment of seismic hazard of Albania (1999-2003) using the probabilistic approach (Aliaj et al., 2010), concluded that the zone with:

- PGA= (0.24-0.28g) covers 1.6% of the Albanian territory,
- 0.20-0.24g →7.2%,
- 0.16-0.20g →32.4%,
- 0.12-0.16g → 47.5%,
- 0.08-0.12g →11.3%.

The highest PGA values are concentrated along the Albanian orogen thrust front, precisely in the Montenegro-Albania border region, in the zone of Vlora and Tepelena, and in the southwestern part of the country. Due to lack of data for the exact location of the precast structures that this study aims to analyze, in this step the selection of the construction site is based on the following three parameters, in order to be enough conservative:

- The building project was designed for the highest seismic zone in Albania, hence from the current map the most active region is chosen;
- On the other hand, the geotechnical parameters of the selected zone should be unfavorable;
- The zone selected should have had a significant economic development in the 1980s, as to presume that this type of structure was constructed there.

Hence in this study an old city that belongs to the Montenegro-Albania border region is chosen and further analyzed, namely Shkodra. During the communist regime many type - buildings were constructed here, from confined masonry to precast panel buildings. Some historical records about the high seismic activity of the city and unfavorable geological conditions are presented here as follows.

Shkodra region with a rich historical culture of construction, unfortunately has been hit several times by moderate to major seismic events. The area has always been one of the most developed regions in the country with the respective infrastructure and urbanization and the seismic events have generated considerable structural damages in this zone. For instance, in 1855 significant structural destruction was the result of some repetitive major seismic events. Not only structures were destroyed from this earthquake, but ground failures were observed as well. These ground failures were not a single time event, as in 1905 when one of the biggest series of strong earthquakes ($M_s=6.6$, $t=10-12\text{sec}$) hit Shkodra, cracks on the ground were observed again. Large structural damage was present, a total of totally destroyed houses and other heavily damaged buildings. This major seismic event caused about 200 dead and about 500 injured (Aliaj et al., 2010).

As noted above, not only the structural damage itself, but the physical-geological phenomenon accompanying the seismic event is a crucial factor to be considered in the hazard analysis of a country. In the Table 4.2 are given some data on liquefaction and other ground phenomena evidenced during some strong earthquakes in Shkoder, Albania (Aliaj et al., 2010).

Table 4.2 Ground phenomena evidenced in some strong earthquakes in Shkoder, Albania (Aliaj et al., 2010)

Date	M	Intensity I_0	Location	Evidenced Phenomena
01/06/1905	6.6	IX	Trush, Bahcallek, Tabak	Subsidence of river banks and ground fissures; water and sand fountains.
17/08/1948	5.5	VIII	Trush,	Ground fissures ($l= 30-50\text{cm}$, $w= 0.5\text{m}$); fountains with sand and water.
15/04/1979	6.9	IX	Montenegro Coast (District of Shkodra)	Intensive subsidence of ground; ground slides; ground fissures ($l= 50-250\text{cm}$, $w=40-50\text{cm}$)& water and sand fountains.

In order to accurately generate a prediction of the seismic hazard of a certain region, the fundamental relation that is needed is the prediction of peak ground motion as a function of earthquake magnitude and distance. Nowadays, these ground motion estimating models are generated from the statistical analysis of strong-motion records available. Unfortunately, in Albania these records are not available, which is the main reason why this country doesn't have an adequate attenuation model yet. In these circumstances neighbor regions models of attenuation are used or models accepted and used worldwide need to be considered. A number of ground-motion relations have been proposed for Europe (Ambraseys, Douglas, Sarma & Smit, 2005; Sabetta & Pugliese, 1996). However, in the cases when strong motion data is missing, not only the geographically nearby positioning of the countries is enough for the selection of attenuation equations, there should be a considerable resemblance between the two countries so that this relation can be applicable in both. In some of the cases, where lesser data is available the empirical relations are developed by pooling some of the data from other regions also (Fukushima & Tanaka, 1990). Data for the probabilistic hazard map in Albania are available in recent studies (Aliaj et al., 2010) but not in the current design code. In order to provide a comparative basis for the values of the peak ground acceleration, in this study the Fukushima & Tanaka model (Fukushima & Tanaka, 1990) is used. Its characteristics are presented as follows in Table 4.3, and marked with tick when applicable for the current case study.

Table 4.3 Characteristics of Fukushima & Tanaka model

Application area	Japan + USA	(Used worldwide)	√
E (number of earthquakes)	28+15	(Our case study 35)	√
M_{scale} (Earthquake magnitude scale)			
Ms (Surface Wave Magnitude) /(M _{JMA})	(Used catalogue has Ms values)		√
M_{min}	4.6 (5)	(Our case M _{min} =5)	√
M_{max}	8.2 (7.7)	(Our case M _{max} =6.9)	√
d_{min} (shortest source- to-site distance)16 km			
d_{max} (largest source- to site distance) 303 km			√
M (Source mechanisms of earthquakes) A (All) (assumed if no information is given in the ref.)			
			√

From the above data compilation, it can be deduced that this equation has high compatible properties with the current case needs. Thus, the current attenuation equation will be used and further developed in this session. The Fukushima & Tanaka equation has the form:

$$\log_{10}A_{max} = 0.41Ms - \log_{10}(R + 0.32(10^{0.41Ms}) - 0.0034R + 1.3 \quad (4.1)$$

Where,

- ✓ **A_{max}** is the peak ground acceleration from two horizontal components
- ✓ **R** is the closest distance
- ✓ **Ms** is surface-wave magnitude

4.5.1 *Earthquake Catalogue*

The database for earthquake data records is the catalogue for earthquakes of magnitude $M_s > 4.5$, for the period 1900-2005 (Aliaj et al., 2010). As described earlier the area selected for analysis is Shkodra, located within these coordinates: 41.87-42.31, 19.39-19.82. In the catalogue all earthquakes that occurred in the period 1900-2005 (which are instrumentally recorded) located within the above defined coordinates are listed in Appendix B. The data provided for these events suggests that:

- ✓ They are generally shallow earthquakes, with depth ranging 10-20 km;
- ✓ Largest earthquake has magnitude $M_s=6.9$;
- ✓ In total they are listed 35 earthquakes.

4.5.2 *Methodology*

The probabilistic methodology is chosen in this section to define the seismic hazard level. Probabilistic Seismic Hazard Analysis (PSHA) defines the probability of exceeding a certain magnitude of earthquake event at a certain region where are given all possible earthquakes. The first analytical methodology to PSHA was introduced by Cornell (Cornell, 1968), and still nowadays it is used to determine the characteristics of ground motion for engineering design. According to Cornell

occurrence of earthquake events are assumed to follow the Poisson's process in time and space, and earthquakes itself are treated as point sources.

The first step is to determine the statistical distribution of earthquakes in time. In the following Table 4.4 it is shown the cumulative frequency of earthquakes recorded in the specified area. Class limits are magnitudes (5, 5.5, 6, 6.5), whereas the frequency graph is plotted in Figure 4.4. *Ms* stands for earthquake magnitude, whereas *N* for the number of earthquakes.

Table 4.4 Earthquake cumulative frequency

Ms	N
5	35
5.5	16
6	6
6.5	2

Gutenberg and Richter have determined the fact that generally the earthquakes follow a particular distribution, given by equation (4.2). Hence, the Gutenberg and Richter law specifically is used to determine the rates at which seismic events of certain magnitudes are expected to occur.

$$\log_{10}N(m) = a - bm \quad (4.2)$$

In this equation $N(m)$ represents the cumulative number of earthquakes characterized by magnitude $M > m$. Whereas a and b are constants, and represent the overall rate of earthquakes in a region and the relative ratio of small and large magnitudes earthquakes, respectively. The estimation of these two constants is crucial to model the seismicity of the zone. This is done by applying the above equation and the estimated values of a and b parameters are found from the linear trendline (regression line) of the following frequency graph Figure 4.4. In 105 years, for the current regression line the recurrence parameters a and b resulted to be 5.735 and 0.831, respectively.

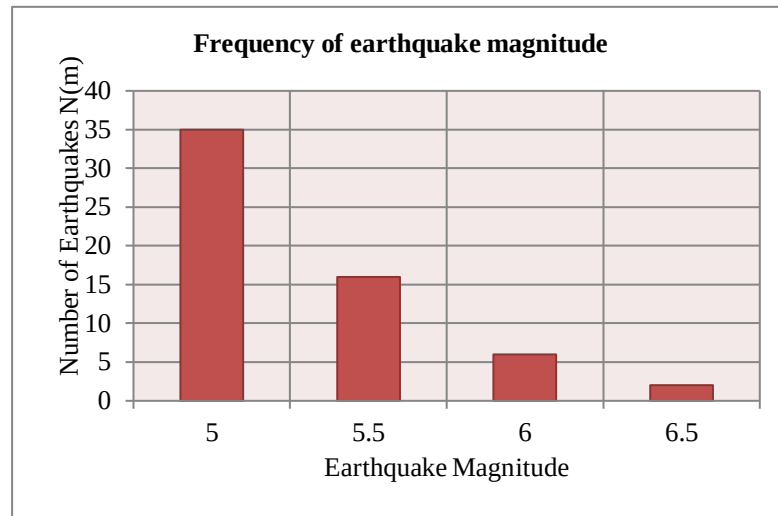


Figure 4.3 Statistical distribution of earthquake magnitude

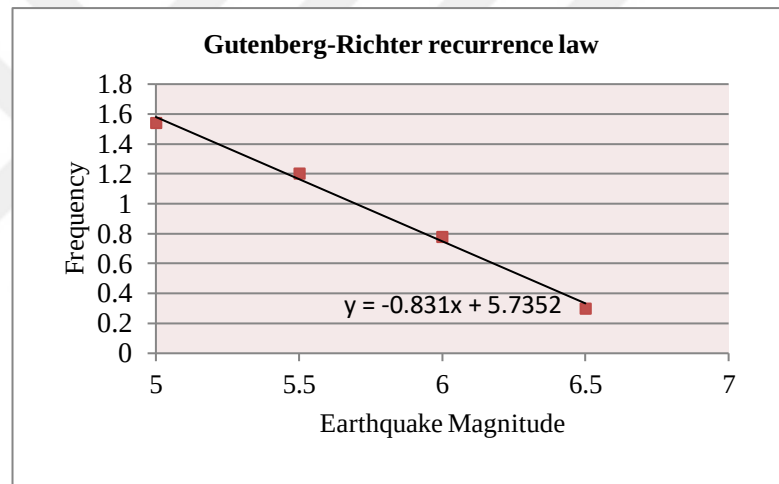


Figure 4.4 Frequency graph – Gutenberg- Richter recurrence law

The period of interest is 1900-2005, thus the results obtained above will be checked with the studies already done to find these recurrence parameters for various periods in Albania (Aliaj et al., 2010). As previously noted, b represents the relative ratio of small and large magnitudes earthquakes, and can be considered as the seismotectonic representative of the region. Using the data for the earthquakes pertaining to the period 1901-1985, it has been found that for $4.6 < M_s < 7.5$, the equation is as follows:

$$\log N = 5.75 - 1.1M_s \quad (4.3)$$

Thus current results are $a=5.735$, $b=0.831$ and the previous results are $a=5.75$, $b=1.1$. The rate terms represent the annual rate of the earthquake scenario. In order to evaluate the mean occurrence rate of earthquakes per year, it is assumed that only earthquakes with magnitudes 5, 5.5, 6 and 6.5 may occur. Therefore, these data are summarized in Table 4.5, as follows:

Table 4.5 Annual rate of the earthquake

In 105 years	Period	Within 1 year
$n(5.0)=38$	1900-2005	$n(5.0)=0.362$
$n(5.5)=14.6$	1900-2005	$n(5.5)=0.139$
$n(6.0)=5.6$	1900-2005	$n(6.0)=0.053$
$n(6.5)=2,15$	1900-2005	$n(6.5)=0.02$

The next step is application of the Second Poisson Assumption which suggests a spatially uniform occurrence of earthquakes within the region. In more technical terms it means that occurrence in space follow a Stationary Poisson Process. So, it is possible to compute the mean annual occurrence rate of earthquakes per unit area of the considered zone. The area enclosed in the coordinates predefined is $1753km^2$. Whereas area ratio is $1/A_T=0.000556$. The mean annual occurrence rate of earthquakes per unit area of the considered zone is presented in Table 4.6.

Table 4.6 Mean annual occurrence rate of earthquakes per unit area

Earthquake Magnitude A	P (annual occurrence) B	Area ratio ($1/A=1/2867$) C	Occurrence (Annual /Unit area) BxC
5.0	0.362	0.000556	0.000201
5.5	0.139	0.000556	7.73E-05
6.0	0.053	0.000556	2.97E-05
6.5	0.02	0.000556	1.14E-05

The main focus of the analysis is the ground motion, not the earthquake itself. Therefore, once the distribution of potential earthquake magnitudes and locations have been defined, the next step is to define a ground motion prediction model. The ground motion model used in PSHA is referred to as an Attenuation Equation, which predicts peak ground motions as a function of earthquake magnitude and distance. As

discussed above, the equation used is Equation 4.1. The equation is plotted, for different magnitudes, as shown in Figure 4.5. Based on the estimated values, it is computed the distance that can produce $A_{max} \geq a$, when magnitude M_s is fixed. Again only magnitudes $M_s=(5; 5.5; 6; 6.5)$ are taken into account. The intersection points needed to produce these distances are shown in Table 4.7.

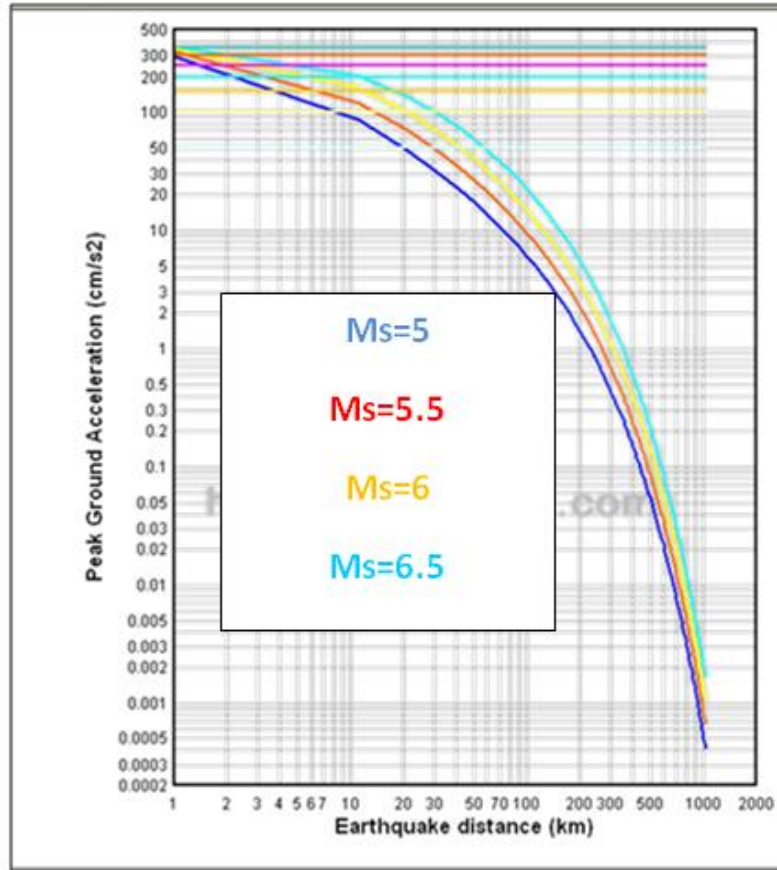


Figure 4.5 PGA relation to earthquake distance

Table 4.7 Corresponding distance to a specific A_{max} and M

A_{max}	$M_s=5$	$M_s=5.5$	$M_s=6$	$M_s=6.5$
50	20.39	29.7	42.06	57.9
100	10.37	15.15	20.93	30.31
150	8.01	9.58	12.78	18.75
200	5.64	7.13	8.86	11.34
250	3.2	4.6	6.1	7.9
300	2.9	2.2	3.41	4.7
350	1.18	1.35	1.61	1.47

Using the radii generated in the previous step, the relative unit area is obtained as $\pi R^2/A_T$, hence the respective values are presented in Table 4.8.

Table 4.8 Area ratio for $A_{max} > a$ with M_s fixed

A_{max}	$M_s=5$	$M_s=5.5$	$M_s=6$	$M_s=6.5$
50	0.7447	1.5800	3.1687	6.0048
100	0.1926	0.4111	0.7846	1.6455
150	0.1149	0.1643	0.2925	0.6297
200	0.0569	0.0910	0.1406	0.2303
250	0.0183	0.0379	0.0666	0.1117
300	0.0150	0.0086	0.0208	0.0395
350	0.0024	0.0032	0.0046	0.0038

The next step is computing the mean annual occurrence of ground motion at the site from the sources with a certain magnitude M_s , when the ground motion is greater than certain acceleration. In order to determine this, the annual occurrence rates of earthquakes are combined with a certain magnitude M_s per unit area and the respective generated values are summarized in Table 4.9.

The relationship between acceleration a (cm/s^2) and the mean annual occurrence rate is called a seismic hazard curve. Therefore, in the last column of Table 4.9 is presented the summation of the mean annual occurrence rate of the ground motion greater than acceleration a (cm/s^2). Instead of the mean annual occurrence rate v , the probabilistic representation is often used based on the following equation (4.4).

Table 4.9 Mean annual occurrence rate for ground motion

A (cm/s^2)	$M=5$	$M=5.5$	$M=6$	$M=6.5$	Total
50	0.2696	0.2180	0.1640	0.1222	0.7740
100	0.0691	0.0567	0.0406	0.0334	0.2
150	0.0412	0.0226	0.0151	0.0128	0.0919
200	0.0204	0.0125	0.0072	0.0046	0.0449
250	0.0065	0.0052	0.0034	0.0022	0.0175
300	0.0054	0.0011	0.0010	0.0008	0.0084
350	0.0008	0.00045	0.00024	7.871E-05	0.00165

$$Prob [A_{max} > a] = 1 - \exp (-v) \quad (4.4)$$

In the forthcoming Figure 4.6, it is presented the hazard curve developed for the peak ground acceleration. The horizontal lines (red, green, purple) plotted represent the annual frequency exceedances and correspond to probabilities typically used for design: return period 72 years, 475 and 2475 years. The peak ground acceleration values corresponding to the probabilities 50%, 10% and 2% in 50 years are generated from the intersection of the seismic hazard curve with the horizontal lines (probabilities of interest), and are summarized in Table 4.10.

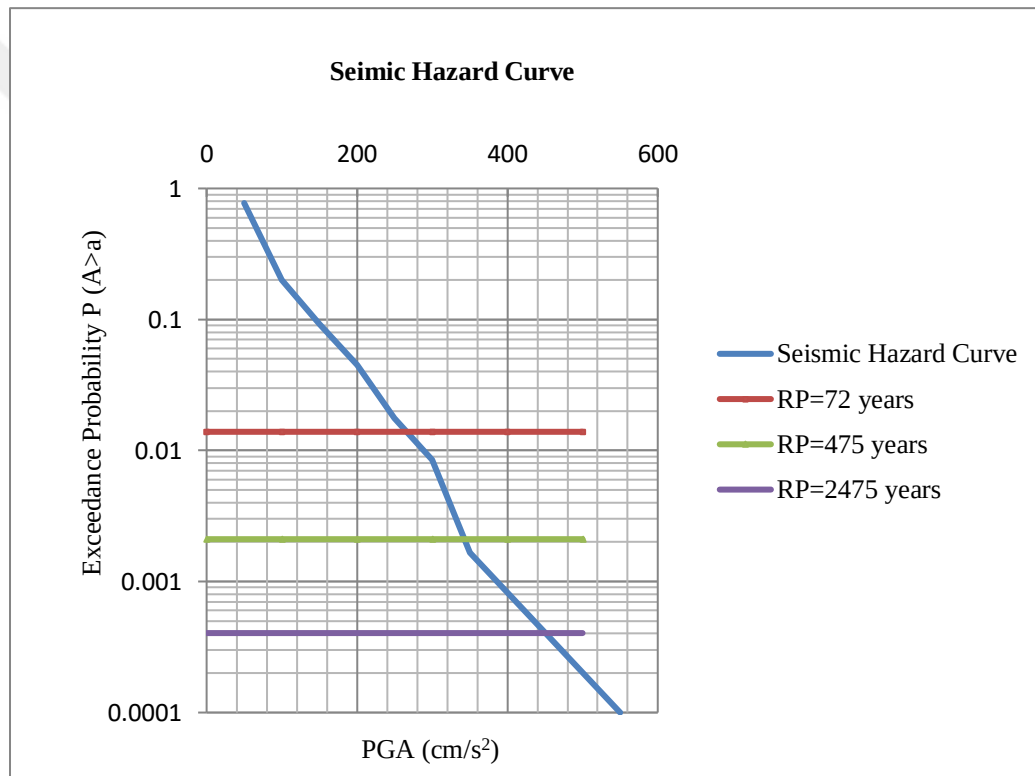


Figure 4.6 Hazard curve developed for peak ground acceleration

Table 4.10 Peak ground accelerations corresponding to respective return periods

Ground Motion	RP=72 years	RP=475 years	RP=2475 years
PGA (cm/s ²)	260	340	450

The values obtained are average for the whole area, with coordinates 19.39-19.82 /41.87-42.31. The first two probabilities can be checked with the peak ground accelerations already provided in the reference material (Aliaj et al., 2010). In the following Table 4.11 are summarized all the regions in the considered area and their peak ground accelerations in terms of the 10% in 50 years probability.

Table 4.11 Peak ground accelerations corresponding to each sub-region

Region	10% in 50 years
1. Ana e Malit	0.394
2. Barbullush	0.394
3. Berdice	0.377
4. Bushat	0.394
5. Dajc	0.394
6. Guri i Zi	0.348
7. Hajmel	0.373
8. Postribe	0.268
9. Pult	0.139
10. Rrethinat	0.309
11. Shale	0.166
12. Shkoder	0.377
13. Shllak	0.261
14. Shosh	0.229
15. Temal	0.261
16. Vau i Dejes	0.346
17. Velipoje	0.397
18. Vig Mnelle	0.324
Average	0.3195

4.5.3 Results

As noticed, the average value of the Peak Ground Acceleration for the 10% in 50 years (0.346) is approximately same as in the reference book (0.32) (Aliaj et al., 2010), yet it is more conservative. The above reported values of PGA are used to develop the 5% damped elastic response spectra for the site into consideration and implemented in the Performance Based Design method.

CHAPTER FIVE

SEISMIC PERFORMANCE EVALUATION

5.1 Introduction

There are several reasons that pose the old precast RC structures in high seismic risk, as mentioned in more detail in previous chapters, such as: poor material properties, non-conforming transverse reinforcement, old design codes and high seismic hazard. According to clause 4.3.3.4.2.1 (3) (EN 1998-1, 2004): “*The prototype building conforms to the regularity criteria in plan and elevation; hence the analysis may be performed using two planar models, one for each main horizontal direction.*” In this case the weakest plane frame is chosen to be analyzed. In this situation, despite the fact that the buildings are regular in plan and elevation, some basic linear procedures are not enough to determine the seismic performance of the structure, its weak points, and the specific demand for retrofit. Hence in this section Nonlinear Static Pushover Analysis is considered appropriate to provide the database necessary for the retrofitting process.

According to Clause 6.6.1.1 in FEMA-356 (2000) “*Precast concrete frames that emulate cast-in-place moment frames shall be defined as those precast beam-column systems that are interconnected using reinforcement and wet concrete to create a system that resists lateral loads in a manner similar to cast-in-place concrete systems*”. The structure under consideration, as described earlier fulfills these criteria. Hence acceptance criteria for precast concrete frames that emulate cast-in-place moment frames shall be as specified in Section 6.5.2.4 (FEMA-356, 2000).

5.2 Evaluation Methodology

The Pushover Analysis keeps track of the sequence of hinges formation after each load step, providing therefore a clear view of the weak parts of the structure in every stage. This nonlinear static procedure is commonly used to estimate the strength capacity of the structure beyond its Limit State up to its ultimate strength. This

analysis method may help the designer predict which members may reach earlier the critical states, progressive failure and the mode of final failure of the structure after a seismic event. The most important step in obtaining an accurate capacity curve for the building under consideration, involves:

- Determination of the way the structural element is expected fail: shear or flexure;
- Determination of the exact position of the expected hinge in the element;
- Determination of the real force-deformation for the hinge region;
- Assign the appropriate acceptance criteria

However, in order to find the performance point of a structure, the demand curve is also quite important. Albania's current seismic map is not accurate enough (Baballëku, 2014) to provide the needed spectra, hence the previous chapter used a probabilistic approach to determine the seismic parameters, which will be used throughout this study. A typical representation of the moment rotation capacity of a hinge is depicted in Figure 5.1.

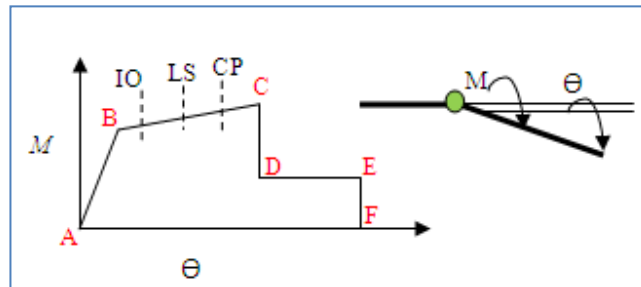


Figure 5.1 Moment-rotation capacity of a typical hinge

- AB is the linear range;
- BC is an inelastic linear response;
- CD is capacity sudden drop;
- DE is the constant reduced resistance;
- EF is the capacity loss.

5.3 Columns - Failure Mode

There is one important check that needs to be done in order to determine the failure mode of the columns. First of all, it is recalled here that the precast joints in the frame under consideration are realized only in the mid-height of the third story. Hence the connection basically is provided in the least stressed region, where no failure is expected. Therefore, the remaining check for the failure mode of the column is: “Will it be a flexural or shear type failure expected in the column span extremities?”.

The presence of increasing shear forces in a region where flexural deformation arises, reduces the flexural deformability of that section. In the plastic hinges a shear capacity decrease has been observed due to the strength degradation of the concrete in this region caused by the flexural cracks. In this situation there is a high chance that the shear (brittle) behavior may dominate the overall behavior of the element, if the shear strength decreases below the flexural strength (Dilsiz, 2013).

For this check it is accepted that for each column in the frame under consideration, at the two extremities of the member are created flexural hinges. Therefore the flexural-type shear capacity (V_e) is calculated according to (TEC, 2007).

$$V_e = \frac{M_u + M_l}{l_n} \quad (5.1)$$

Where: - M_u / M_l are the moment capacities at upper / lower ends of the column
 - l_n is the length of the column.

The shear-type failure capacities (V_r) of the columns were calculated according to (TS-500, 2000).

$$V_r = V_c + V_w = 0.8V_{cr} + \frac{A_{sw}}{s f_{yw} d} \quad (5.2)$$

Where:

- V_c is the capacity from concrete
- V_w is the capacity from transverse reinforcement
- A_{sw} is total area of transverse reinforcement within a distance of s
- f_{ywd} is design yield strength of the transverse reinforcement
- d is distance from extreme compression fiber to the centroid of longitudinal tension reinforcement
- V_{cr} is the cracking shear strength of the cross-section

$$V_{cr} = 0.65f_{ctd}b_wd \left(1 + \gamma \frac{N_d}{A_c}\right) \quad (5.3)$$

Where:

- f_{ctd} is design tensile strength of the concrete
- b_w is the width of the cross-section
- d is distance from extreme compression fiber to the centroid of longitudinal tension reinforcement
- N_d is axial load
- A_c the concrete area in cross-section.

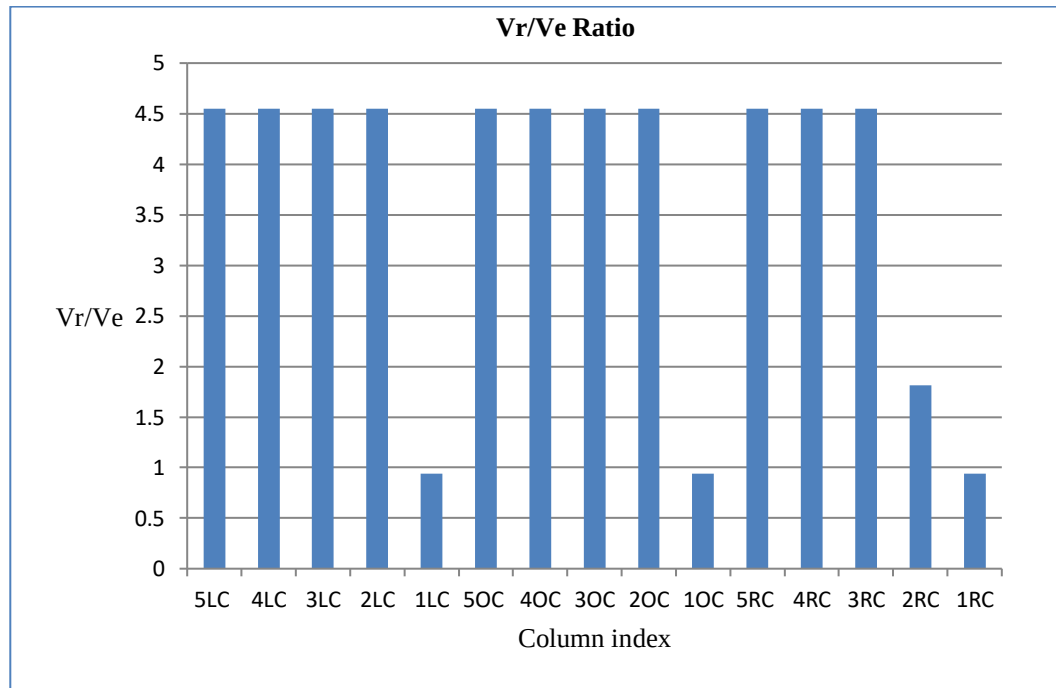


Figure 5.2 Vr / Ve ratios for each column

The calculated shear capacity values, using equations (5.1) and (5.2) were compared with each other by means of the V_r / V_e ratios, in order to determine the mode of failure of the columns (Dilsiz, 2013), and are shown in Figure 5.2.

The mode of failure for the columns having a V_r / V_e ratio which is lower than 1.0, is expected as brittle shear-type failure. However, in this case, most of the columns show a ratio more than 1. Only 3 columns have a ratio slightly less than 1(0.96). This occurred because when considering the cracking shear strength of the cross section, in order to be conservative, it is neglected the positive effect of the axial compression such as $(1 + \gamma * N_d / A_c)$, where γ is taken as 0.07 considering the axial compression on columns. It can be concluded here, that all the columns are expected to show a flexural type failure, and this is how they are modeled for the pushover analysis.

5.4 Beam - Failure Mode

In Chapter 3 the beam precast joint type and location is described in more detail. The conclusions drawn so far are resumed as follows:

- It allocates the yielding beyond the connection region; hence it can be considered as a strong connection. In these conditions again the yielding is expected in the region between the column face and the precast joint.
- In this case the strong connection is located in a considerable distance from the column face, such that yielding is expected to spread in the beam up to the column face and not have concentrated damage.

5.5 Case Study- Hinge Generation

In order to obtain an accurate capacity curve, all the hinges in this section were analytically determined, based on the flexural type failure of both beam and columns and their cross-section properties. At first the yielding and ultimate moment of the

section were determined. The excel-sheet provided by the authors of the book (Chandrasekaran, Nunziante, Serino & Carannante, 2010), were deemed valuable to obtain these results. Then the yield and ultimate curvature and rotation of each section were derived based on (EN1998-3, 2005). Here a brief review of the main formulas needed to get the necessary database is presented.

$$\text{Yield rotation: } \theta_y = \Phi_y \frac{L_v + a_v z}{3} + 0.0014 \left(1 + 1.5 \frac{h}{L_v} \right) + \Phi_y \frac{d_{bL} f_y}{8 \sqrt{f_c}} \quad (5.4)$$

Where (EN1998-3, 2005):

- Φ_y is the yield curvature of the end section
- $a_v z$ is the tension shift of the bending moment diagram
- z length of internal lever arm
- $a_v = 1$
- f_y and f_c are the yield stress and the concrete strength
- ϵ_y is equal f_y/E_s
- d and d' are the depth to the tension and compression reinforcement
- d_{bL} is the mean diameter of the tension reinforcement

$$\text{Ultimate rotation: } \theta_u = \frac{1}{\gamma_{el}} \left[\theta_y + (\Phi_u - \Phi_y) L_{pl} \left(1 - \frac{0.5 L_{pl}}{L_v} \right) \right] \quad (5.5)$$

Where (EN1998-3, 2005):

- θ_y is the cord rotation at yield
- Φ_y is the yield curvature of the end section
- Φ_u is the ultimate curvature of the end section

For members with detailing for earthquake resistance and without lapping of longitudinal bars in the vicinity of the section where yielding is expected, L_{pl} may be calculated from the following equation 5.6 (EN1998-3, 2005):

$$L_{pl} = 0.1 L_v + 0.17 h + 0.24 \frac{d_{bL} f_y (MPa)}{\sqrt{f_c (MPa)}} \quad (5.6)$$

$$L_v = M/V \quad (5.7)$$

5.5.1 Beam Hinges - Analytical Approach

In the Figure 5.3 all the cross sections of the beams in the frame with the respective reinforcement details are presented. Then the hinges parameters database for these sections is shown in Table 5.1.

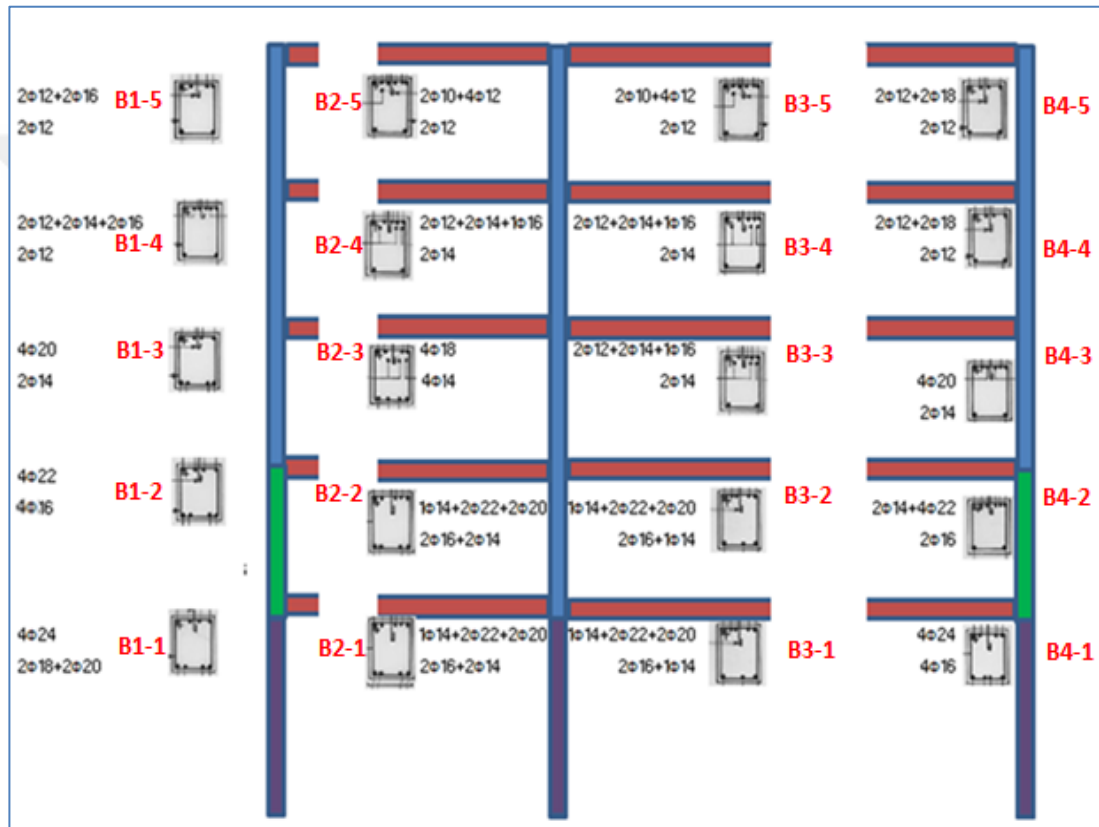


Figure 5.3 Beam cross-sections and reinforcement details

A typical property of the hysteresis needs to be highlighted in this step: in many beam sections it is observed that there is a significant difference between the top and bottom reinforcement, hence depending on the loading condition that will generate in a specific section either a negative or positive moment, the proper hinge should be determined. Therefore, the beam hinges are not symmetric.

In some certain cases when there is significant change between the top and bottom reinforcement of the beams, after some loading steps hinge problems may arise. This considerable difference in reinforcement may lead to formation of nonreversible hinges, which might cause damage concentration in one region. Hence these cases should be carefully considered.

Table 5.1 Hinge parameters for beams

Section	Φ_y (rad/m)	Φ_u (rad/m)	Θ_y (rad)	Θ_u (rad)	M_y (kN)	M_u (kN)
B1-5	0.00355	0.03	0.00443	0.01056	-37.817	-40.715
B1-5	0.00283	0.0282	0.00387	0.00988	14.764	15.130
B1-4	0.00395	0.0316	0.00475	0.01112	-54.559	-59.261
B1-4	0.00275	0.0282	0.00381	0.00986	14.756	15.116
B1-3	0.00435	0.033	0.00506	0.01161	-74.828	-80.793
B1-3	0.0028	0.0283	0.00385	0.00990	20.164	20.493
B1-2	0.00455	0.0345	0.00521	0.01209	-90.067	-96.858
B1-2	0.0032	0.0288	0.00416	0.01014	51.687	53.178
B1-1	0.00423	0.0316	0.00497	0.01118	-111.506	-118.528
B1-1	0.0034	0.029	0.00432	0.01024	72.908	74.865
B2-5	0.00357	0.03	0.00445	0.01057	-38.009	-40.715
B2-5	0.00282	0.0283	0.00387	0.00990	14.711	15.130
B2-4	0.00365	0.0302	0.00451	0.01064	-43.629	-46.691
B2-4	0.00293	0.0284	0.00395	0.00996	20.003	20.521
B2-3	0.0038	0.0303	0.00463	0.01071	-62.830	-67.114
B2-3	0.0032	0.0289	0.00416	0.01017	39.599	40.795
B2-2	0.00427	0.032	0.00500	0.01131	-92.388	-98.923
B2-2	0.00312	0.0287	0.00410	0.01009	45.627	46.769
B2-1	0.00427	0.032	0.00500	0.01131	-92.388	-98.923
B2-1	0.00312	0.0287	0.00410	0.01009	45.627	46.769
B3-5	0.00357	0.03	0.00508	0.01206	-38.009	-40.715
B3-5	0.00282	0.0283	0.00435	0.01126	14.7119	15.130
B3-4	0.00365	0.0302	0.00516	0.01216	-43.629	-46.691
B3-4	0.00293	0.0284	0.00445	0.01133	20.003	20.521
B3-3	0.00365	0.0302	0.00516	0.01216	-43.629	-46.691
B3-3	0.00293	0.0284	0.00445	0.01133	20.003	20.521

Table 5.1 continues

Section	Φ_y (rad/m)	Φ_u (rad/m)	Θ_y (rad)	Θ_u (rad)	M_y (kN)	M_u (kN)
B3-2	0.00443	0.0335	0.005929	0.01349	-91.309	-97.857
B3-2	0.00298	0.0286	0.004508	0.01141	34.575	35.184
B3-1	0.00443	0.0335	0.005929	0.01349	-91.309	-97.857
B3-1	0.00298	0.0286	0.004508	0.01141	34.575	35.184
B4-5	0.0037	0.0305	0.005214	0.01227	-43.298	-46.597
B4-5	0.0028	0.0282	0.004332	0.01122	14.751	15.125
B4-4	0.0037	0.0305	0.005214	0.01227	-43.298	-46.597
B4-4	0.0028	0.0282	0.004332	0.01122	14.751	15.125
B4-3	0.00435	0.033	0.005851	0.01330	-74.828	-80.793
B4-3	0.0028	0.0283	0.004332	0.01126	20.164	20.493
B4-2	0.0049	0.036	0.006389	0.01447	-105.562	-112.931
B4-2	0.0028	0.0282	0.004332	0.01122	26.308	26.687
B4-1	0.0045	0.033	0.005998	0.01335	-109.804	-116.725
B4-1	0.00315	0.0287	0.004675	0.01150	52.139	53.168

Despite the analytical hinge parameters generation, in this section another check was done for the database obtained referring to FEMA-356 (2000). In this code there are tables suggesting the plastic rotation parameters for concrete beams and columns, with the respective acceptance criteria. In order to define the category in which the beams fall, the following checks are done:

- Transverse reinforcement is every 25cm ($>d/3$), hence non-conforming
- At the last pushover step most of beams fall in the category $V/b_w d \sqrt{f'c} > 6$

Therefore, the selected case is: Beams controlled by flexure; $A=0.005$, $b=0.01$, $c=0.2$, $IO=0.0015$, $LS=0.005$ and $CP=0.005$. The parameter a suggested by (FEMA-356, 2000) is 0.005, whereas from the analytical procedure the values range from 0.006-0.007, hence the analytical approach showed a slightly larger plastic deformation capacity of the hinges. Hence in the following Table 5.2 the modeling parameters for the hinge's plastic rotation are presented.

Table 5.2 Modeling parameters for beam plastic hinges

Beam	a	b	C	Beam	a	b	C
<i>B1-5</i>	-0.00613 0.006004	-0.01 0.01	-8.143 3.026	<i>B3-5</i>	-0.00698 0.006916	-0.01 0.01	-8.143 3.026
<i>B1-4</i>	-0.00637 0.006048	-0.01 0.01	-11.852 3.023	<i>B3-4</i>	-0.007 0.006877	-0.01 0.01	-9.338 4.104
<i>B1-3</i>	-0.00655 0.00605	-0.01 0.01	-16.158 4.098	<i>B3-3</i>	-0.007 0.006877	-0.01 0.01	-9.338 4.104
<i>B1-2</i>	-0.00687 0.005974	-0.01 0.01	-19.371 10.635	<i>B3-2</i>	-0.00757 0.00691	-0.01 0.01	-19.571 7.036
<i>B1-1</i>	-0.00622 0.005922	-0.01 0.01	-23.705 14.973	<i>B3-1</i>	-0.00757 0.00691	-0.01 0.01	-19.571 7.036
<i>B2-5</i>	-0.00612 0.006039	-0.01 0.01	-8.143 3.0261	<i>B4-5</i>	-0.00706 0.006896	-0.01 0.01	-9.319 3.025
<i>B2-4</i>	-0.00613 0.006007	-0.01 0.01	-9.338 4.104	<i>B4-4</i>	-0.00706 0.006896	-0.01 0.01	-9.319 3.025
<i>B2-3</i>	-0.00608 0.006003	-0.01 0.01	-13.422 8.159	<i>B4-3</i>	-0.00746 0.006929	-0.01 0.01	-16.158 4.098
<i>B 2-2</i>	-0.00631 0.005989	-0.01 0.01	-19.784 9.353	<i>B4-2</i>	-0.00808 0.006896	-0.01 0.01	-22.586 5.337
<i>B2-1</i>	-0.00631 0.005989	-0.01 0.01	-19.784 9.353	<i>B4-1</i>	-0.00736 0.006831	-0.01 0.01	-23.345 10.633

A typical beam hinge model in SAP2000, 2007 used for the pushover analysis is shown in Figure 5.4.

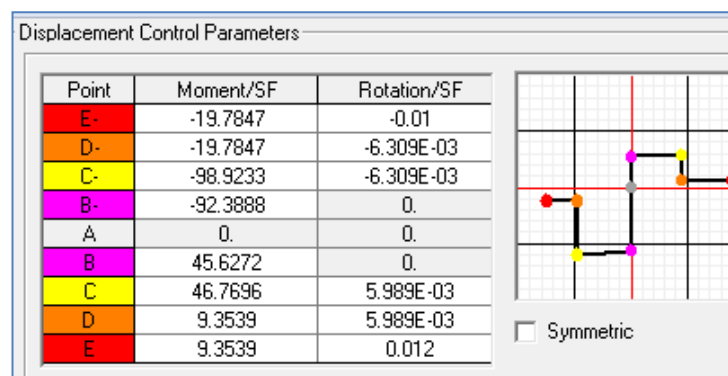


Figure 5.4 A typical beam hinge model in SAP2000

5.5.2 Column Hinges - Analytical Approach

In Figure 5.5 all the cross sections of the columns in the frame with the respective reinforcement details are presented. Then the hinges parameters database for these sections is shown in Table 5.3. For the columns the P-M3 hinge pattern is activated. Hence in this case also the moment-axial force interaction diagram is needed, and is presented in Table 5.4. Differing from the beam case, for the column hinges a symmetric moment-rotation is defined.

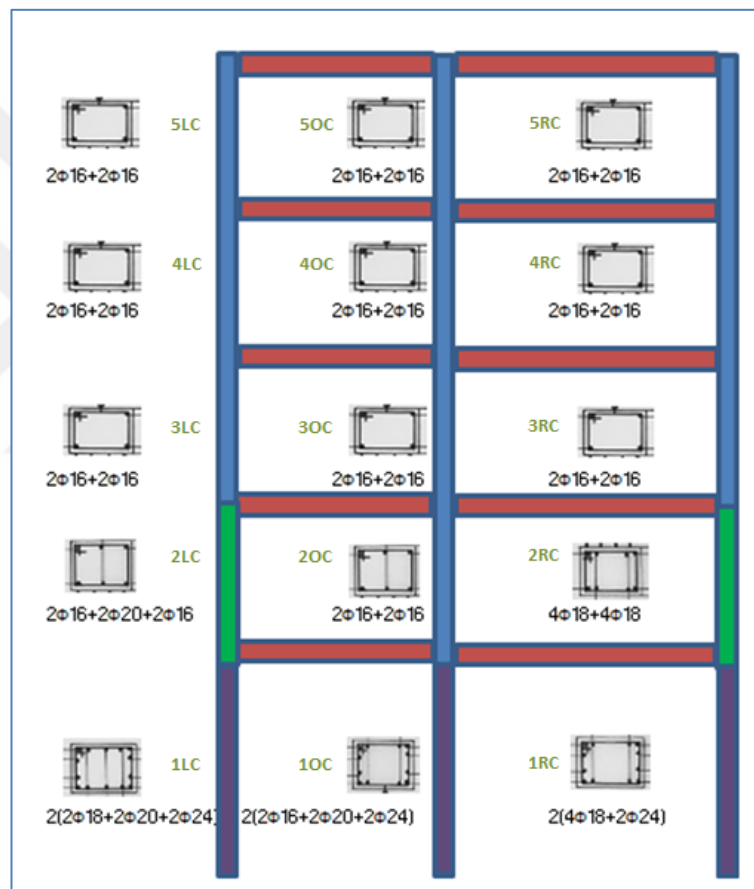


Figure 5.5 Column cross-section and reinforcement details

Table 5.3 Hinges parameters database for columns

Section	Φ_y (rad/m)	Φ_u (rad/m)	Θ_y (rad)	Θ_u (rad)	M_y (kN)	M_u (kN)
5LC	0.00317	0.0289	0.003625	0.00851	25.631	26.764
4LC	0.00317	0.0289	0.003625	0.00851	25.631	26.764

Table 5.3 continues

Section	Φ_y (rad/m)	Φ_u (rad/m)	Θ_y (rad)	Θ_u (rad)	M_y (kN)	M_u (kN)
3LC	0.00317	0.0289	0.003625	0.00851	25.631	26.764
2LC	0.00317	0.0289	0.003625	0.00851	25.631	26.764
1LC	0.00393	0.03	0.004049	0.00888	124.405	129.316
5OC	0.00317	0.0289	0.003625	0.00851	25.631	26.764
4OC	0.00317	0.0289	0.003625	0.00851	25.631	26.764
3OC	0.00317	0.0289	0.003625	0.00851	25.631	26.764
2OC	0.00317	0.0289	0.003625	0.00851	25.631	26.764
1OC	0.00393	0.03	0.004049	0.00888	124.405	129.316
5RC	0.00317	0.0289	0.003625	0.00851	25.631	26.764
4RC	0.00317	0.0289	0.003625	0.00851	25.631	26.764
3RC	0.00317	0.0289	0.003625	0.00851	25.631	26.764
2RC	0.0036	0.03	0.003865	0.00883	64.314	67.221
1RC	0.00393	0.03	0.004049	0.00888	124.405	129.316

Table 5.4 Moment-axial force interaction values for columns

5LC	P (kN)	-147	572	1350
	M (kNm)	0	85	0
4LC	P	-147	572	1350
	M	0	85	0
3LC	P	-147	572	1350
	M	0	85	0
2LC	P	-147	572	1350
	M	0	85	0
1LC	P	-743	521	1894
	M	0	192	0
5OC	P	-147	572	1350
	M	0	85	0
4OC	P	-147	572	1350
	M	0	85	0
3OC	P	-147	572	1350
	M	0	85	0
2OC	P	-147	572	1350
	M	0	85	0
1OC	P	-702	572	1876
	M	0	185	0
5RC	P	-147	572	1350
	M	0	85	0

Table 5.4 continues

4RC	P	-147	572	1350
	M	0	85	0
3RC	P	-147	572	1350
	M	0	85	0
2RC	P	-371	546	1560
	M	0	126	0
1RC	P	-702	572	1876
	M	0	185	0

The corresponding graphs for the moment - axial load interaction diagram for the representative cases are presented in the Appendix C. Despite the analytical hinge parameters generation, in this section another check was done for the database obtained referring to (FEMA-356, 2000), as previously done for beams. In order to define the category in which the columns fall, the following checks are done for “Columns controlled by flexure”:

Condition 1: $0.1 < P/Ag f'_c < 0.4$

Condition 2: Transverse reinforcement is every 25cm ($> d/3$), hence Non-conforming

Condition 3: At the last pushover step most of columns fall in the category $V/b_w d \sqrt{f'_c} > 6$

Hence the selected case is: Columns controlled by flexure, $a=0.005$, $b=0.01$, $c=0.2$, $IO=0.005$, $LS=0.005$ and $CP=0.005$. The parameter a suggested is 0.005, as well as from the analytical procedure the values are approximately 0.005, hence the two cases are compatible. In the following Table 5.5 the modeling parameters for the hinge’s plastic rotation are presented:

Table 5.5 Modeling parameters for the column plastic hinges

Section	A	B	C
5LC	0.00489	0.012	5.3528
4LC	0.00489	0.012	5.3528
3LC	0.00489	0.012	5.3528
2LC	0.00489	0.012	5.3528
1LC	0.00483	0.012	25.863
5OC	0.00489	0.012	5.3528

Table 5.5 continues

Section	A	B	C
4OC	0.00489	0.012	5.3528
3OC	0.00489	0.012	5.3528
2OC	0.00489	0.012	5.3528
1OC	0.00483	0.012	25.8632
5RC	0.00489	0.012	5.3528
4RC	0.00489	0.012	5.3528
3RC	0.00489	0.012	5.3528
2RC	0.00497	0.012	13.444
1RC	0.00483	0.012	25.863

A typical column hinge model in SAP2000, 2007 used for the pushover analysis is shown in Figure 5.6.

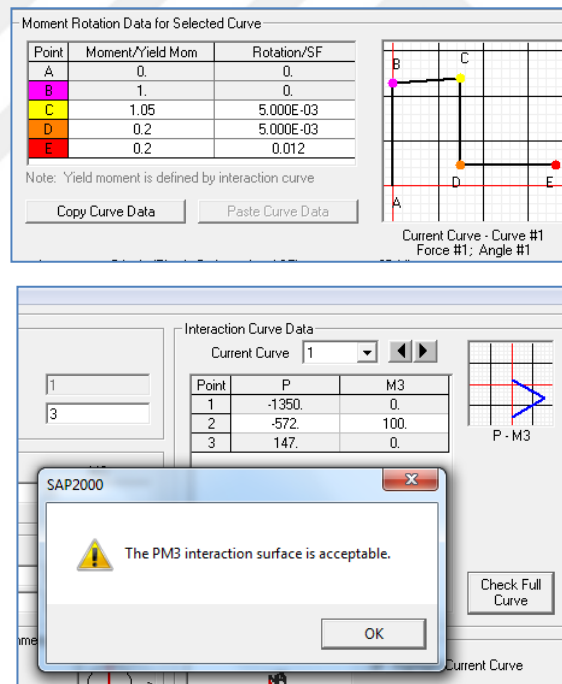


Figure 5.6 Moment-rotation and moment-axial load curve for column hinge

The overall hinge assignment in the columns and the beams of the frame in SAP2000 are shown in Figure 5.7.

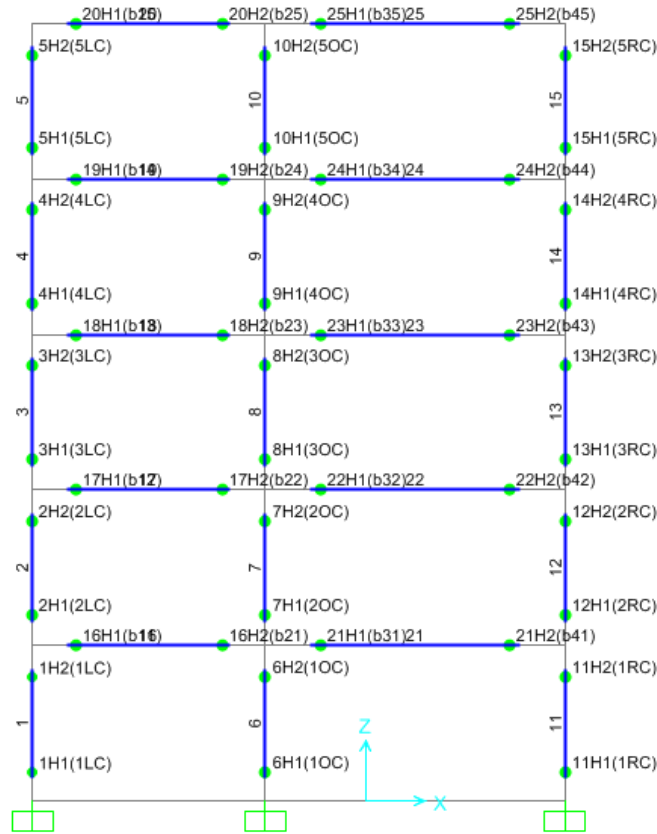


Figure 5.7 Frame hinge assignment

5.6 Pushover Analysis Parameters

Analytical research results that were reflected in some of the most important retrofitting design codes (ATC, 1996) state that for structures dominated by first mode response (which is the case for the structure under consideration), the capacity spectrum methodology provides a good average estimate of the displacements predicted by multiple time history analyses using different records with the same enveloping spectrum. The pushover analysis procedure is described in brief highlighting the main parameters input, whereas a more detailed behavior of both the original frame and the strengthened one with the time history analyses results are provided in following chapters.

a) Material properties

- Beams and columns are constructed with concrete of grade M300 – which is equivalent to class C23 concrete (Kadiu, 2004)

- $f_y=210\text{MPa}$
 - Concrete cover is 2cm;
- b) Restraint support** – Fixed
- c) Floor diaphragm** – Assigned to each level
- d) Mass source** – From element and additional mass and loads (Dead +0.3 Live)
- e) Period** - $T=0.65\text{ s}$
- f) Hinge unloading method** – Unload entire structure
- g) Structural Behavior Types** - Choose average existing building for short shaking duration, hence Type B. Note: This parameter is important when defining the performance point.
- h) Soil type** – is unknown => choose Type D (as suggested by ATC, 1996)
- i) Control displacement** - The control displacement may be taken at the center of mass of the roof of the building.
- j) Behavior factor for precast structures**

According to clause 5.11.1.4 (1) EN1998-1 (2004) for precast structures, the values of the behavior factor q_p may be calculated from the following Equation 5.8, unless special studies allow for deviations:

$$q_p = k_p q \quad (5.8)$$

Where, q is the behavior factor, k_p is the reduction factor depending on the energy dissipation capacity of the precast structure. The lower bound limit is considered $q_p=k_p \quad q=0.5 \quad 3*1.3=2$ and the maximum value $q_p=k_p \quad q=3*1.3=3.9$, therefore by interpolating between the values $q=3$ is selected.

k) Lateral loads pattern

- According to 4.3.3.4.2.2 (1) EN 1998-1 (2004), at least two vertical distributions of the lateral loads should be applied:
 - “Uniform” pattern, based on lateral forces that are proportional to mass regardless of elevation (uniform response acceleration);

- “Modal” pattern, proportional to lateral forces consistent with the lateral force distribution in the direction under consideration determined in elastic analysis.
- (2P) Lateral loads shall be applied at the location of the masses in the model. Accidental eccentricity in accordance with 4.3.2 (1) P shall be taken into account.

Both, the uniform and the modal pattern are presented in Table 5.6.

Table 5.6 Uniform and modal loading pattern

	U1 (modal)	Uniform pattern	Modal pattern (U_i/U_5)
<i>Floor 1</i>	0.0237	1	0.20413
<i>Floor 2</i>	0.0553	1	0.47631
<i>Floor 3</i>	0.0834	1	0.71834
<i>Floor 4</i>	0.1041	1	0.89664
<i>Floor 5</i>	0.1161	1	1

5.7 Pushover Analysis Results

As the first step the pushover or capacity curve needs to be generated. The process is crucial to keep track of the critical points in the structure step by step, and moreover is independent of the method used to calculate the demand. In technical terms pushover curve is base shear (sum of all horizontal support reactions in that direction) vs roof displacement at the centre of mass of the roof. Pushover procedures are classified as: Displacement coefficient method and capacity spectrum method.

DCM (Displacement Coefficient Method) before performing the analysis estimates a Target Displacement, to which the model has to be pushed, and then checks for the desired performance at that displacement. Whereas CSM (Capacity Spectrum Method) when performing the analysis, consecutively checks whether Sa-

S_d at the point of interest meets the Demand Curve, which is obtained after reduction by a factor. Capacity Curve and Demand curve are consecutively checked together for any intersection. Performance Point is actually the point where these two curves intersect.

5.7.1 Case 1- Uniform Load Pattern

As previously stated, the frame is loaded with two patterns. In this section the first pattern, namely the uniform one is analyzed. At first the hinge distribution pattern is analyzed. In the following pictures, a clear view of the most critical points in the structure is provided. The dots color legend representing different stages of the hinge development for beams and columns are previously shown in this section in Figure 5.4 and Figure 5.6 respectively.

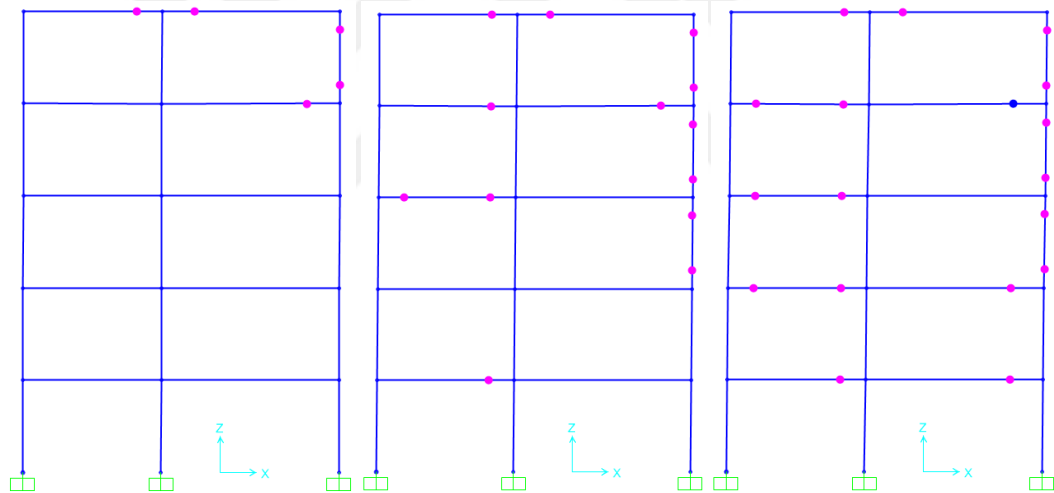


Figure 5.8 Hinge pattern: Stage 1-3

In the first pushover stages (Figure 5.8) hinges are distributed mainly in the beams and in the right column. A major presence of hinges in the beams, when compared to columns, shows that a desired type of behavior is currently present at this first stage. However, no hinges are present in the first stories columns as required in normal hinge development mechanisms. This is due to the high difference in the reinforcement ratios between lower and upper stories. This phenomenon is more obvious in subsequent stages.

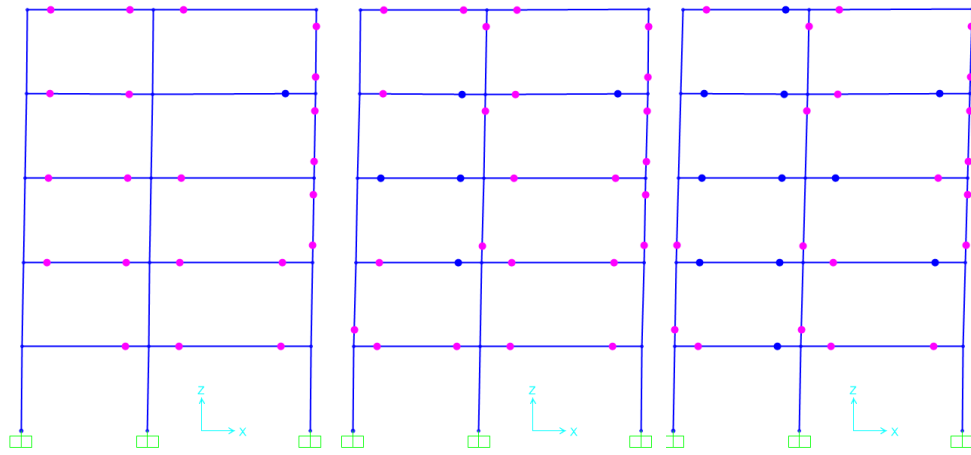


Figure 5.9 Hinge pattern: Stage 4-6

In the second set of stages (Figure 5.9), hinges continue to add in the beam ends, and also start to appear in the middle column. The column hinges are mostly accumulated in the upper stories, which is expected from the significant lower reinforcement ratios they have when compared to the first stories.

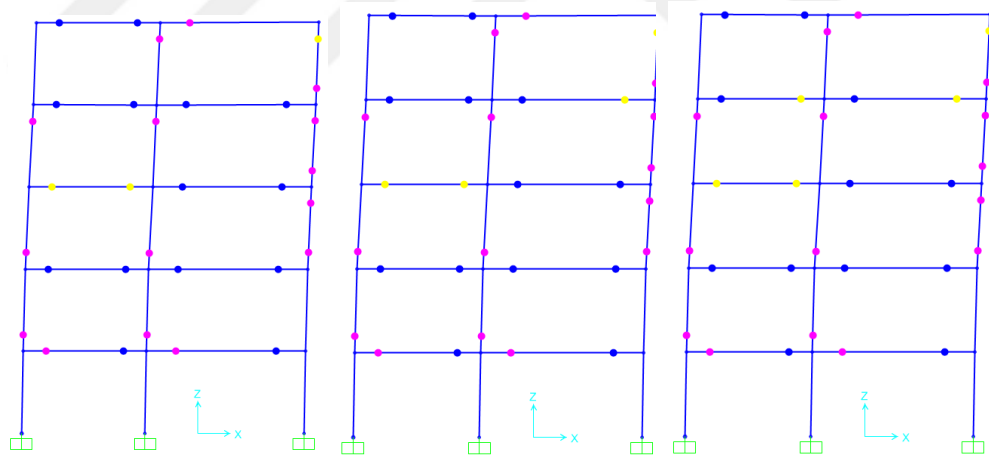


Figure 5.10 Hinge pattern: Stage 7-9

In the third set (Figure 5.10) it is observed that in one column and one beam, the hinges have just reached the limit rotation for the full capacity, once this limit is passed their capacity is drastically reduced. Again, it is interesting to note that in the first story columns where the maximum stress is expected, no hinges have formed yet, which as previously noted can be attributed to the high reinforcement ratios present in these columns.

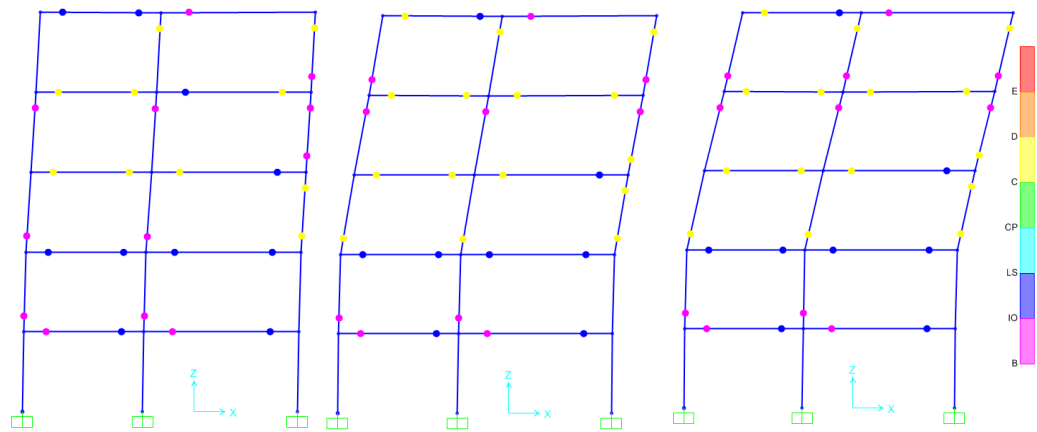


Figure 5.11 Hinge pattern: Stages 10-12

In the last set (Figure 5.11) more hinges have reached the limit rotation for full capacity. Moreover, in the last two stages it is clearly visible that there is a concentrated drift in the upper stories, creating a soft third story. The pushover curve obtained for this structure is presented in Figure 5.12. The main phase changes that can be observed are noted under the red arrows. These arrows shown in the graph, correspond to the respective phases:

- Initial yielding of the frame
- Initiation of strength degradation
- 20% decrease of capacity
- 50% decrease of capacity

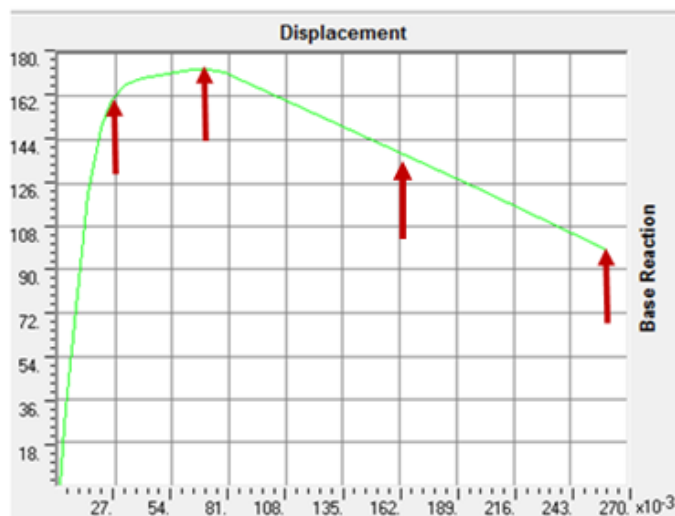


Figure 5.12 Base shear (kN) vs roof displacement (m)

Interstory drifts for each main phase as defined above, are shown in Table 5.7. The initiation of frame yielding occurs at step 4. This can be attributed to the hinges pattern. In this step almost all beams have hinges that have reached the yield rotation capacity, whereas in step 10, the frame starts to show a capacity degradation. The hinge pattern suggests that 10 hinges have just reached the limit rotation for the full capacity, and after this phase their capacity degrades. The formation of soft stories as obtained from the hinges pattern, is also obvious from Table 5.7 in the last column of data that belongs to step 12.

Table 5.7 Displacement (m) and interstory drifts

Floor	d (m) @ Step 4	Drift	d (m) @ Step 10	Drift	d (m) @ Step 12	Drift
<i>Floor 5</i>	0.025	0.00146	0.0787	0.0064	0.2605	0.0290
<i>Floor 4</i>	0.0209	0.00196	0.0607	0.008	0.1792	0.0310
<i>Floor 3</i>	0.0154	0.00210	0.0381	0.008	0.0922	0.0286
<i>Floor 2</i>	0.0095	0.00196	0.0157	0.0036	0.0121	0.0028
<i>Floor 1</i>	0.004		0.0056		0.004	

The performance of the building is evaluated for the design earthquake. The strength and deformation limits recommended for this level of performance are selected based on the target objective in Table 5.8.

Table 5.8 Target objective for structure performance

Performance objective				
	Building performance level			
EQ-Ground Motion	Operational	Immediate occupancy	Life safety	Structural stability
Serviceability EQ				
Design EQ			√	
Maximum EQ				

In the previous section the pushover curve was obtained, whereas in this section the demand curve is generated. The seismic zone factor Z , accounts for the amount of seismic risk present in a zone based on the anticipated maximum ground acceleration at the site.

The zone factor of 0.4 in zone 4 represents the maximum base unadjusted effective peak acceleration is $0.4g$ for the *design earthquake*. According to (ATC, 1996) recommendation, when the soil properties are not known in sufficient detail to determine the Soil Profile Type, Type SD may be used. Hence for the site seismicity characteristics, selection is done as following:

- From ATC, 1996 Table 4-4 choose $Z=0.3$
- For the design earthquake:
 - $C_A=0.36$ from ATC, 1996 Table 4-7
 - $C_V=0.54$ from ATC, 1996 Table 4-8

Once the demand spectra are defined the performance point can be obtained as shown in Figure 5.13. This point has the respective coordinates (121.757; 0.203). This result shows that the current demand imposes considerably high drifts in the structure, hence in order to properly define the structure performance at this level a general view of the hinges deformation levels corresponding to this step is provided next.

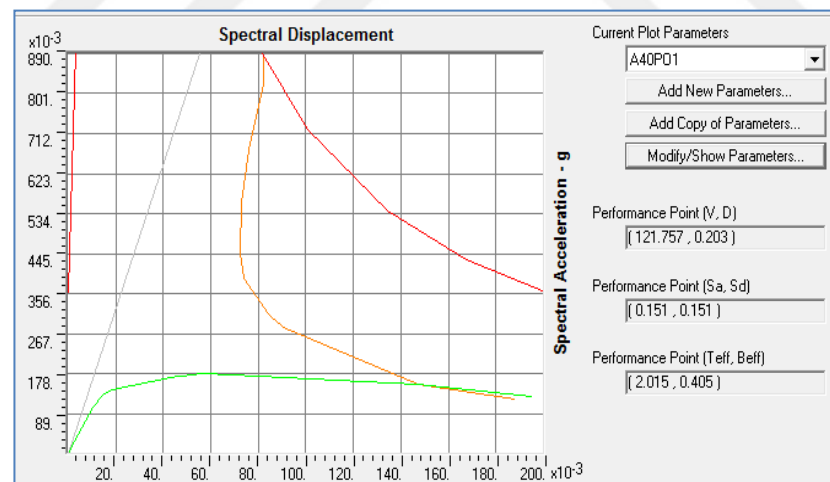


Figure 5.13 Performance point of the structure

The performance point corresponds to step 11. To this frame 50 hinges have been assigned, from which:

- 10 (20%) hinges have just reached the yield rotation capacity

- 8 (16%) hinges have just passed in the plastic rotation phase, but within the immediate occupancy level
- 15 (30%) hinges have reached the ultimate rotation for full capacity, hence the collapse prevention level
- 8 out of 15 hinges that have reached the collapse prevention level are concentrated in the upper story columns, by creating a soft story in the third floor.
- Lateral deformations at the performance point displacement are checked against the deformation limits provided by ATC, 1996 Table 5.9. Maximum total drift – the interstory drift at the performance point displacement, is 0.021-0.023 for the 3 upper stories, hence more than the limit suggested.

Table 5.9 Global acceptance criteria: deformation limits provided by ATC, 1996

Interstory drift limit	Performance Level			
	Immediate occupancy	Damage control	Life safety	Structural stability
Maximum total drift	0.01	0.02	0.02	0.33Vi/Pi
Maximum inelastic drift	0.005	0.005-0.015	No limit	No limit

Moreover, the lateral load resistance of the building system, including resistance to the effects of gravity loads acting through lateral displacements, should not degrade by more than 20 percent of the max resistance of the structure according to clause 2.4.1 Response Limits of ATC, 1996. When the frame has reached the roof displacement 0.16m, hence 1.1% total drift, the frame has lost 20% of its capacity. Hence the frame has lost significant strength, before reaching the performance point.

5.7.2 Case 2 – Modal Load Pattern

In this section the second load pattern is applied to the frame, in order to obtain its performance. As a first step the hinge distribution pattern is analyzed. In the following pictures, a clear view of the most critical points in the structure is provided.

The dots color legend representing different stages of the hinge development for beams and columns are previously shown in this section in Figure 5.4 and Figure 5.6 respectively.

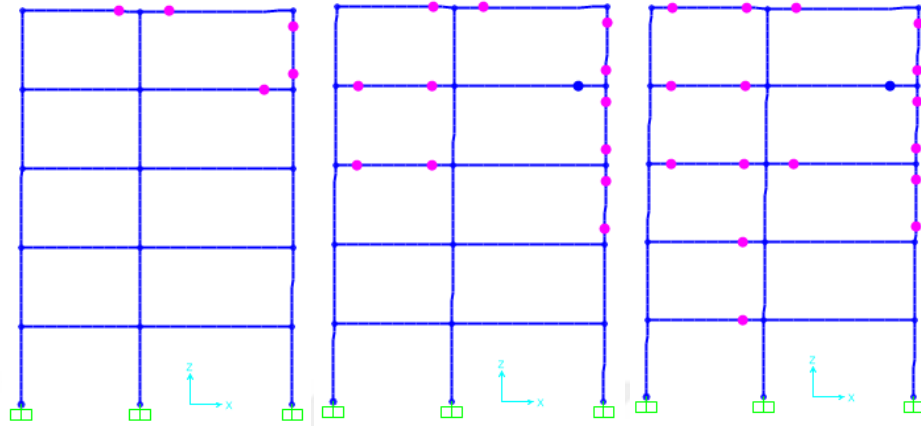


Figure 5.14 Hinge pattern: Stages 1-3

In the first pushover stages (Figure 5.14) hinges are distributed mainly in the beams and in the right column, hence showing a desired type of behavior. It is highlighted here that the hinges pattern is quite similar to the case of uniform loading, as expected.

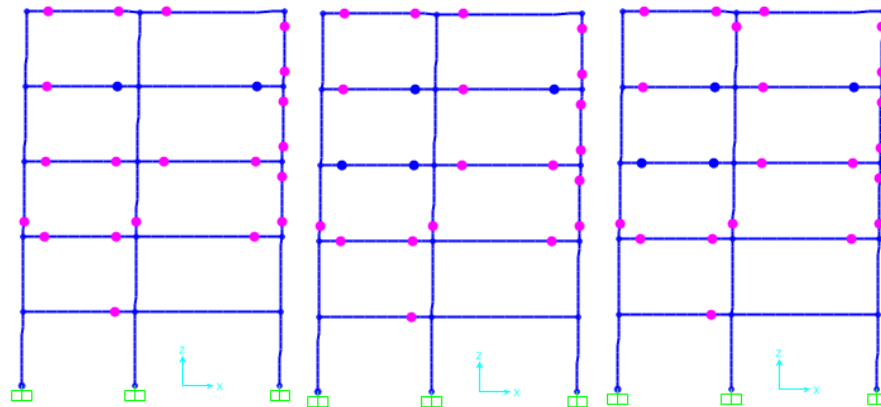


Figure 5.15 Hinge pattern: Stages 4-6

In the second set of stages (Figure 5.15), hinges continue to add in the beam ends. The column hinges are mostly accumulated in the upper stories, which is expected from the significant lower reinforcement ratios they have when compared to the first two stories.

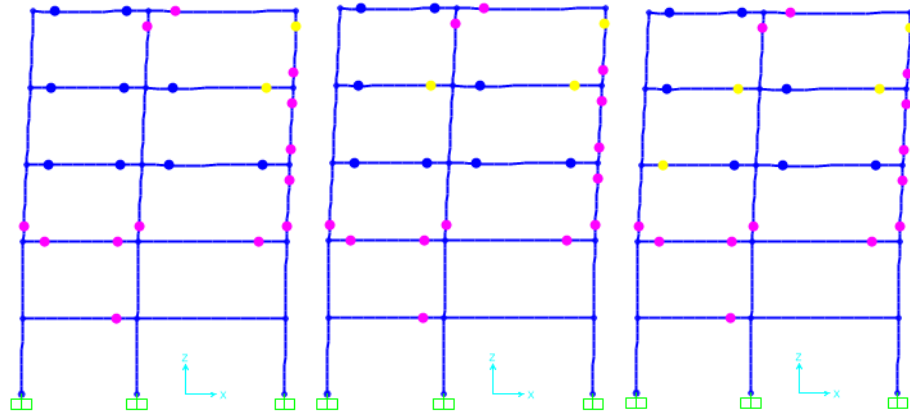


Figure 5.16 Hinge pattern: Stages 7-9

In the third set (Figure 5.16) it is observed that up to 4 hinges (3 hinges in beam and 1 hinge in column) have just reached the limit rotation for the full capacity (the yellow hinges). Once this limit is passed their capacity is drastically reduced. Column hinges are accumulated in the upper stories, which is attributed to the considerably lower reinforcement ratios present in these columns. This pattern is identical to the case when the loading pattern was uniform.

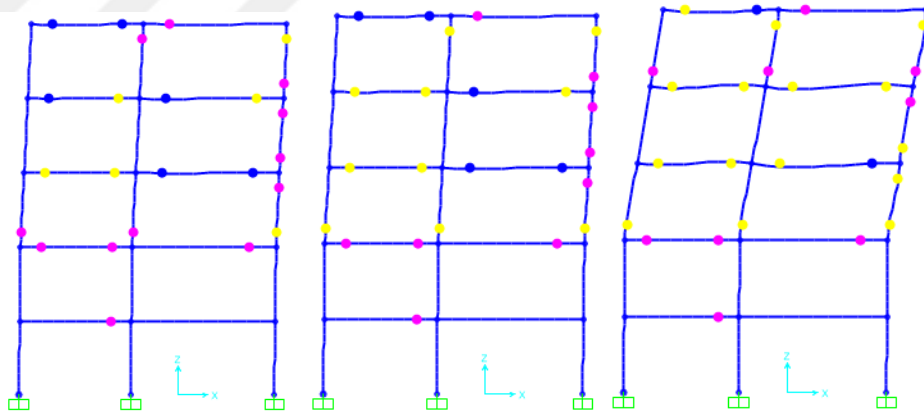


Figure 5.17 Hinge pattern: Stages 10-12

In the last set (Figure 5.17) more hinges have reached the limit rotation for full capacity. Moreover, in the last two stages it is clearly visible that there is a concentrated drift in the upper stories, creating a soft third story. The pushover curve obtained for this structure is presented in Figure 5.18. The main phase changes that can be observed are noted under the red arrows. These arrows are shown in the graph (Figure 5.18) and correspond to the respective phases:

- Initial yielding of the frame
- Initiation of strength degradation
- 20% decrease of capacity

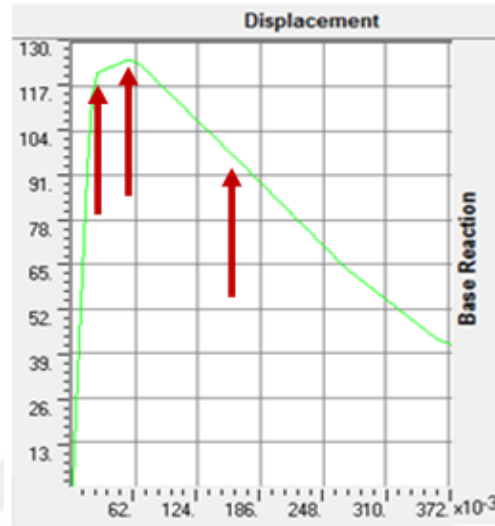


Figure 5.18 Base shear (kN) vs roof displacement (m)

Interstory drifts for each main phase as defined above, are shown in Table 5.10. The initiation of frame yielding occurs at step 3. This can be attributed to the hinges pattern. In this step almost all beams have hinges (pink dots) that that have reached the yield rotation capacity, whereas in step 8 the frame starts to show a capacity degradation. The hinge pattern suggests that some hinges in beams and columns have just reached the limit rotation for the full capacity, and after this phase their capacity degrades. The formation of soft stories as obtained from the hinges pattern, is also obvious from Table 5.10 in the last column of data that belongs to step 12.

Table 5.10 Displacement (m) and interstory drifts

Floor	d (m) @ Step 3	Drift	d (m) @ Step 8	Drift	d (m) @ Step 12	Drift
<i>Floor 5</i>	0.0197	0.001429	0.0562	0.005464	0.164	0.01882
<i>Floor 4</i>	0.0157	0.001714	0.0409	0.006143	0.1113	0.01964
<i>Floor 3</i>	0.0109	0.001607	0.0237	0.005786	0.0563	0.01796
<i>Floor 2</i>	0.0064	0.001321	0.0075	0.001607	0.006	0.00128
<i>Floor 1</i>	0.0027		0.003		0.0024	

The performance of the building is evaluated for the design earthquake. The strength and deformation limits recommended for this level of performance are selected based on the target objective in Table 5.9. Once the demand spectra is defined, it is observed that in this case of demand and capacity no intersection is found Figure 5.19. The frame has lost most of its lateral resistance before any performance point is found. When the frame has reached the roof displacement 0.15m, hence 1.07% total drift, it has lost 20% of its capacity. This result is similar to the previous pushover curve for the uniform load pattern.

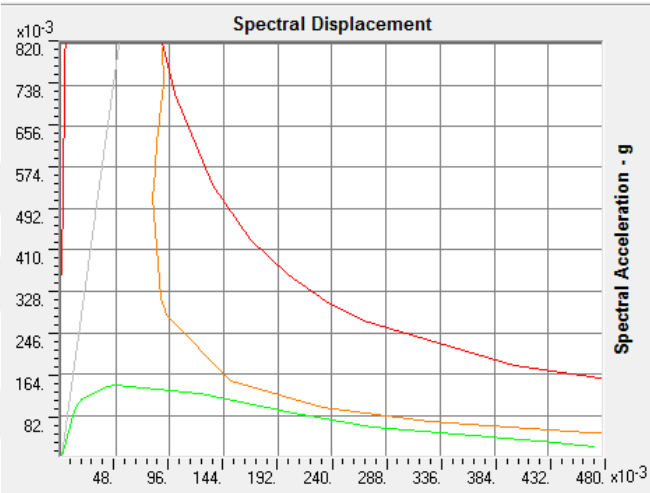


Figure 5.19 Performance point of the structure

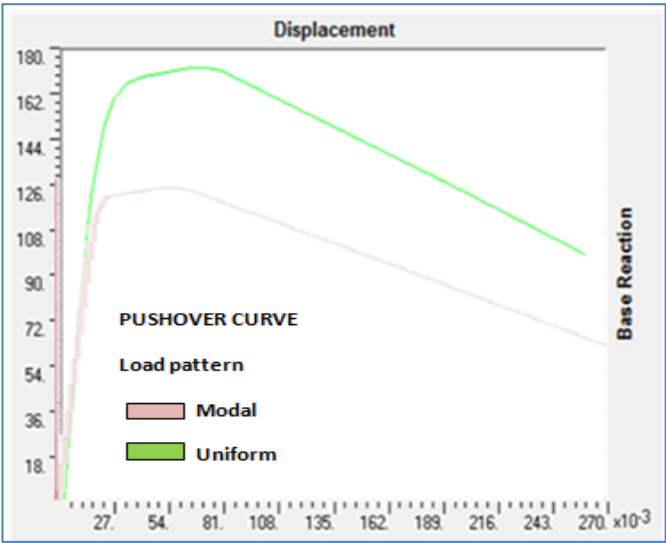


Figure 5.20 Base shear (kN) vs roof displacement (m) for both loading patterns

After providing the pushover results for the two load patterns, as shown in Figure 5.20, it is concluded that these two methods produce different base shear capacities for the frame. This divergence has been encountered also in previous research (Bento, Falcao & Rodrigues, 2004; Etedalli & Irandegani, 2015; Repapis, 2016). However, in this case, probably the divergence in the base shear is due to the fact that, in the modal loading pattern the upper story elements are loaded much more than the base stories, where in fact the upper stories are the least reinforced and provide the lowest capacity. However, both pushover results provide some important common properties, such as:

- Both load patterns yield the same initial stiffness of the frame.
- Frame yielding starts at similar displacement of the last floor, respectively 0.0197m vs 0.025m.
- Both frames have lost more than 20% of their lateral resistance before the performance point is reached.
- At the last cycles, formation of soft story at the same level starts.

Pushover results for both loading patterns of the frame show that: for the provided demand, there is not enough strength and deformation capacity in the frame to obtain at least a life safety performance level as required. In these circumstances retrofit of the structure is needed.

CHAPTER SIX

UFP - FLEXURAL BASED BRACE DEVELOPMENT

6.1 Optimal Retrofit Systems

The capacity curves generated in Chapter 5, with the respective performance points and frame hinges patterns, depict the general performance of the structure when subjected to a design earthquake. It can be concluded that there are two main deficiencies, namely insufficient strength and non-uniform stiffness distribution in height. The structure loses more than 20% of its lateral resistance before the performance point is reached, whereas the soft story mechanism is observed in the third floor. In this situation a retrofit procedure that enhances both strength and assures a uniform stiffness distribution is needed.

Many techniques are available for seismic upgrading of RC frames structures in order to increase capacity of the structure, or/and reduce the seismic demand (Abou-Elfath, Ramadan & Alkanai, 2017; Barbagallo, Bosco, Marino & Rossi, 2017; Cardone & Flora, 2016; Chuang, Tsai, Lin & Wu, 2015; Corte, D'Aniello & Landolfo, 2014; Gong, 2017; Khoo et al., 2016; Lin, Tsai, Wu, Chuang & Li, 2015; Mahrenholtz et al., 2015; Okada, 1995; Pan, Wu, Tsai, Li & Khoo, 2016; Tremblay, Poncet, Bolduc, Neville & DeVall, 2004; Wu et al., 2017). In the large set of strengthening options, two main alternatives are considered here as appropriate for the current case, namely shear walls and bracing systems.

Shear walls are actually one of the most commonly used strengthening scheme, as they can considerably enhance both strength and stiffness of the retrofitted structure. They are generally economical and compatible with most existing concrete structures (ATC, 1996). However, there are some disadvantages of this retrofit alternative that are highlighted here.

- Strengthening can cause a considerable seismic force increase, due to shear wall extra mass added to the original structure.

- Loss of windows and addition of barriers within areas of floor space can arise.
- They generate large overturning forces at their bases requiring additional foundation work, which is quite difficult and expensive.

Braces are characterized by excellent strength and ductility levels as well and provide an effective way to enhance the building strength and stiffness. They generally generate lower levels of stiffness and strength than do shear walls, but their mass addition is considerably lower when compared to shear walls. They can be constructed by causing less function disruption to the building. However, the main drawback of this technique is the selection of the appropriate connection method with the existing frame, as large forces must be transferred from the braces to the existing structure. Once the positive and negative properties of both systems have been summarized, are recalled here the specific requirements that the structure poses when the retrofit alternative has to be chosen. Namely:

- The structures are of residential function, and the occupation cannot be disrupted so a fast application technique should be chosen. Moreover, as the traffic patterns have already been set in the structure, the retrofit scheme should not cause full closure of any frame in case shear wall is chosen.
- In order to optimize the strengthening scheme, uniform stiffness enhancing is not required through the height of structure, but should target the last three floors where the soft story mechanism is observed.
- The best alternative should be to strengthen the structure, without causing any additional large stress in the foundation, in order to avoid their strengthening as well. Foundation strengthening is expensive and cumbersome and might make the retrofit alternative ineligible, whether the strengthening is managed by the local governmental units or by the individuals living in the residential building.

For all the above-mentioned requirements, that aim the optimal strengthening scenario, it is concluded that “braces” provide the best option. However, the strengthening system that shall involve braces, should provide for the structure enough energy dissipation even at small drifts, in order to reduce the displacement demands of the structure in the soft stories.

6.2 Alternative Brace Generation

Buckling restrained braces (BRBs) characterized by their high strength, ductility and ample energy dissipating properties can reduce demand in other structural components. In this perspective they can even avoid drift concentration at certain stores. Their fuse-like action when dissipating energy can yield a favorable collapse mechanism for the structure. The size of BRBs can be optimally selected to provide the required stiffness, without over-sizing the additional lateral strength (Barbagallo et al., 2017). So far, designers have developed these components so that enough strength and ductility is provided by them.

However, there are still some characteristics of this energy dissipating system that need to be improved, when considering that it is required to apply them as a retrofit scheme to a precast old RC structure that has low ductility, such as:

- ✓ initial stiffness;
- ✓ low yield point
- ✓ axial elongation capacity
- ✓ symmetric hysteresis
- ✓ redundancy

6.2.1 Initial Stiffness

The initial stiffnesses of both UFP and BRB braces are presented as follows:

Initial stiffness for UFP brace (k_{tot})

$$k_{tot} = nk_0 = \frac{F_y}{\Delta_y} = \frac{16Eb_u}{27\pi} \left(\frac{t_u}{D_u} \right)^3 \quad (6.1)$$

Where: n =Number of UFP, k_0 = initial stiffness for one UFP, k_{tot} = initial stiffness of brace, b_u = Width of UFP plate section, t_u = Thickness of UFP plate section, D_u = Diameter of UFP bend and E = Elastic modulus

Initial stiffness for BRB (K_{br})

$$K_{br} = \frac{P}{\Delta} \quad (6.2)$$

$$\Delta = \frac{PL_y}{A_y E} \quad (6.3)$$

$$L_y = (0.5 - 0.8) L \quad (6.4)$$

(depending on brace type and configuration)

$$K_{bry} = (1.2 - 2.0)A_y E / L \quad (6.5)$$

Where: K_{br} =BRB initial stiffness, P =Axial load, Δ = Axial deformation, L_y = Brace yield length and L = Brace work point length

Both braces initial stiffness depend on material elastic modulus and the dimensions of the material that is designated to yield, respectively: the number and UFP dimensions in the first case and the core plate section in the second. However there is one main difference here, that the BRB initial stiffness is inversely proportional to brace length, whereas for a given number of UFPs, the UFP brace initial stiffness does not depend on the brace length.

Why this converts to a disadvantage for the BRB brace?

Lets' s assume that in order to obtain the necessary yield load and ultimate load for the brace, it is chosen a convenient number of UFPs for the first brace, and the cross section dimensions for the second brace. In this case for any brace length needed, the first brace initial stiffness is only dependent on the UFP number and despite the length change it remains constant. Whereas in the case of a BRB brace, with increasing the brace length demand the stiffness decreases as well. This means that even if the cross section is enough to provide the yield and ultimate load, it has to be increased in case a specific stiffness for longer braces is needed, either the length of internal sections has to be changed accordingly. Specifically, BRBs with identical specified strengths may have stiffnesses that vary by as much as 20% due to differences in the overall length or the length of internal sections of the brace, which may result in unintentional system behavior (Robinson & Black, 2011). Whereas UFP braces have an initial stiffness dependent just in UFP number and dimensions, and not the brace length.

6.2.2 Low Yield Material

In this study, it is aimed to strengthen an RC frame, whose ductility is limited, hence different criteria from moderate ductile structures should be used in this case when selecting the brace material. So far, the core plate material used for BRB was commonly building steels. For instance, Q235, with nominal yield strength of 235MPa, were widely used in China (Ma, Ou & Li, 2012). This material is suitable for braces incorporated in steel frames with large ductility. However, for the reinforced concrete frames that possess small ductility, in which BRBs are required to enter the inelastic phase at small displacement, the common building steels cannot meet the need. Moreover, if the core plate is made of common steel, it can be used to control the damage in the structure only in case of considerably strong seismic events.

Whereas, when UFP are used for brace, the yielding occurs at very low displacement, no matter what the number of UFPs is involved in the brace. When it is considered the normal brace length 5-7m and the UFP material and configuration

that was previously experimentally tested (Baird, Smith, Palermo & Pampanin, 2014), the brace yields at ~5mm displacement, a range of tensile strain 0.07% to 0.1%, which means that these braces are activated quite early, even in buildings with low ductility.

6.2.3 Axial Elongation Capacity

In order to overcome the problems that normal steel BRB caused, like late activation for non-ductile structures or even non-activation in minor seismic events, researchers have experimented with new materials such as steel with low-yield strength to fabricate the core plate. Several works have been done (Maeda, Nakata & Iwata, 1998; Matteis, Landolfo & Mazzolani, 2003; Saeki, Sugisawa, & Yamaguchi, 1995; Susantha, Aokia & Kumano, 2005) the results have proved that BRBs with core plates made by low-yield strength present good energy dissipation properties under low stress.

However, there was always one main drawback in these systems, as the maximum axial strain amplitude in most of these studies was 1.5%, which was not enough to use the full ductility capacity of these low yield strength steel braces. Whereas, in case UFPs are used to develop a brace, as experimental results suggest (Baird et al., 2014), at 6.3% maximum strain of the UFP plate, the brace could possibly reach up to 80mm axial deformation. For a range of braces length 5m or less, this deformation capacity is equivalent to more than 1.6% axial strain.

However, these results were obtained for steel material with nominal yield strength of 320MPa, hence in case steel with lower yield point is used higher axial strains may be attainable. Moreover, since the maximum strain is also directly related to the geometry of the UFP, the design can also be made such that the UFP can undergo considerable deformations from multiple earthquake events by considering low cycle fatigue criteria (Baird et al., 2014).

6.2.4 Symmetric Behavior-Hysteresis

It is well known that in BRB braces due to friction forces arising in the brace, the compression strength is larger than the tensile strength. This ratio is limited however by the (AISC, 2005) to a level of 1.3. However, this difference might have structural behavior effects, due to this non-symmetrical behavior. In order to provide a comparison between the UFP brace and the common BRB, reference is done to experimental result (Baird et al., 2014; Zhou, Xie, Lei, He & Meng, 2015). The former shows a ratio of 1.09 whereas the latter shows a ratio of 1.19. Hence it can be concluded that the UFP brace has slightly better performance in terms of symmetric behavior of the hysteresis.

6.2.5 Redundancy

The distribution of yielding regions in a member is the main parameter showing that this element will have redundant behavior when subjected to loading. Here are recalled two systems types for comparison in the yielding pattern. The former provides energy dissipation from the core plate yielding in tension and compression (Zhou et al., 2015) whereas the second dissipates energy by yielding in flexure (Baird et al., 2014).



Figure 6.1 Stress concentration in (a) Core plate (b) UFP plate (Baird et al., 2014)

In the first case, after loading fractures are observed in the core plates. Multiple-wave buckling occurred during compression, accompanied by friction between the core plates and the inner and outer tubes (Zhou et al., 2015) (Figure 6.1). Whereas in the case of a UFP, when one of its legs is subjected to a displacement relative to the

opposite leg, the semi-circular section rolls along the plate and work is done at the two points where the radius of curvature changes. Thus, the yielding point of the plate is moved along the plate (Baird et al., 2014). Hence a brace composed of UFP sets will have distributed yielding regions in all elements, and the yielding point within the UFP itself moves along the plate. This is a more favorable behavior, when compared to multi-wave buckling occurring in the core plates of common braces.

6.3 U- Shape Flexural Plate Working Mechanism

The steel U-shaped flexural plate (UFP), initially proposed by (Kelly, Skinner & Heine, 1972), consists of one semicircular part and two straight legs, as shown in Figure 6.2. The relative movement between these parallel legs results in the flexural deformation of the UFP unit. Ultimate displacements up to 85mm were experimentally achieved in a typical UFP test with no decrease in strength (Baird et al., 2014). A design flexibility for this system, is that it can be designed for a large range of possible displacements and force levels by varying the plate dimensions. Moreover, this unit can be bolted to the structural member, so that if damaged it can be replaced after an earthquake. This dissipater is cheap to fabricate, robust, easy to apply and replaceable. Recently it was successfully applied also as an energy dissipation system in shear walls (Baird, Palermo & Pampanin, 2013; Palermo, Pampanin, Buchanan & Newcombe, 2005) and seismic base isolated structures (Jiao, Kishiki, Ene & Yamada, 2014; Konishi, Kawamura, Terashima & Kishiki, 2012; Suzuki, Watanabe & Saeki, 2005).

Recalling here the working mechanism of the self-centering braces, their telescope-like behavior may benefit from the energy dissipation produced from a set of UFPs attached to the tubes moving axially relative to one another. Therefore, in this section a set of UFPs is analyzed as a possible flexure-based energy dissipating mechanism for the braces. At first the basic hysteretic properties of the UFP unit are analytically evaluated, before incorporating this unit in the macro system.

6.3.1 Configuration and Geometry

A mini-brace with two sets of UFPs is analyzed with LS DYNA (Appendix D) Software (LSTC 2007) in order to obtain the fundamental properties of its hysteretic behavior. The steel plate has cross section dimensions of 8x100 mm, total height of 220mm and is bent with a radius of 60mm, as depicted in Figure 6.2. Inner and outer tubes are modeled as rigid parts, due to the small ratio length/width of the brace. L1 is the unconstrained length of the UFP, whereas L2 is the length of the UFP leg that is constrained to the inner and outer tube. Meshing was generated with 4mm tetrahedral elements.

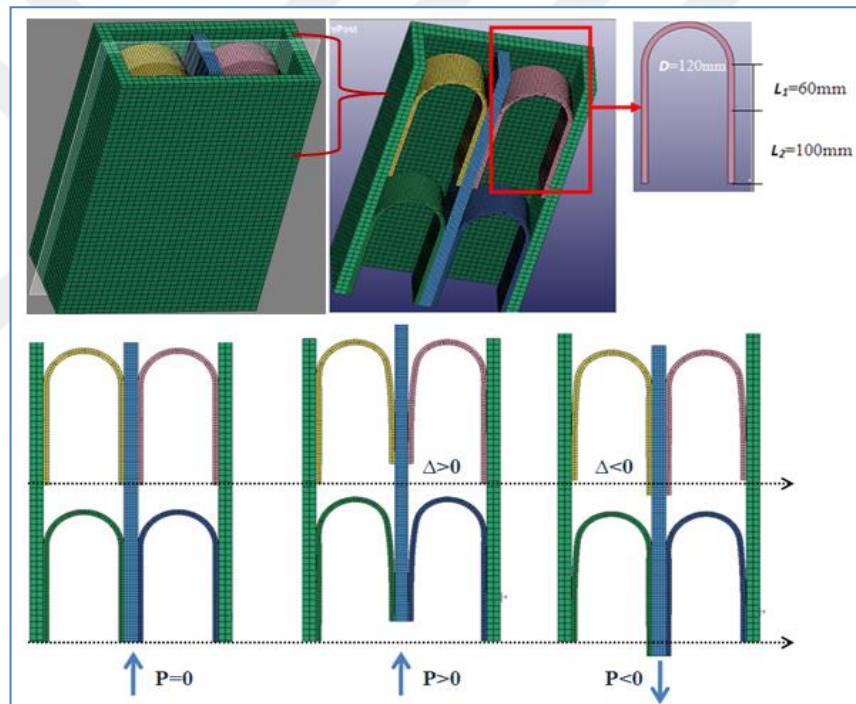


Figure 6.2 Mini brace with two sets of UFPs (Configuration and loading)

6.3.2 U- Shape Flexural Plate Material

Selection of the optimal material for the UFP unit production, poses two essential requirements such as cost effectiveness combined with acceptable mechanical properties, namely strength and ductility. Acceptable candidates for the UFP element generation are both mild and stainless steel. The material modulus of stainless steels

reduces with increasing stress, unlike that of carbon steels which is constant. Moreover, the residual stresses arising from fabrication are higher in stainless steel than in carbon steels (Yoshida & Uemori, 2002). On the other hand, mild steel shows ductile behavior as annealed or at elevated temperature. Finally, when using stainless steel, the dissipater is more expensive and its behavior becomes dependent on strain history due to its isotropic hardening, hence mild steel is preferred.

Therefore, the material chosen is JIS SS400, with initial yield stress 255 MPa. Modeling cyclic plasticity of mild steel is of great importance in the analysis of steel UFP plates subjected to cyclic loading. The Yoshida-Uemori model (Yoshida & Uemori, 2002) is reported to have a high capacity of simulating metals under cyclic loading within large plastic strain ranges (around 20%) (Shi, Zhu, Xia & Stoughton, 2008).

Hence MAT_KINEMATIC_HARDENING_TRANSVERSELY_ANISOTROPIC was chosen as reasonably appropriate material to model the UFP plate with LSDYNA software. It integrates MAT 37 with Yoshida two surface kinematic hardening model. The material properties input in the numerical simulation are summarized in Table 6.1 referring to the previous research database (Jia & Kuwamura, 2013).

Table 6.1 Material parameters input in LS DYNA

Parameter	Value
E:=Young's Modulus.	200GPa
PR:=Poisson's ratio.	0.3
CB:=Initial size of the bounding surface (B*)	277 MPa
Y:=Initial yield stress (σ_{y0} *)	255 Mpa
SC:=Material parameter (C*)	338 MPa
K:=Material parameter to describe the hardening rate of the bounding surface (m*)	7.9
RSAT:=Maximum change in size of the bounding surface (Rsat*)	196 MPa
SB:=Anisotropic parameter associated with work-hardening stagnation (b*)	195 MPa
H:=Formulation option (h*)	0.5

* Respective material parameter symbols in the reference

6.3.3 Loading Regime

In order to determine the ultimate target displacement of the alternative brace, assumption is made that the brace is installed at a 30° angle in an actual frame, for a target interstory drift of 2%. In the previous research (Zhou et al., 2015), a displacement-controlled loading protocol with equal amplitude incremental displacements was used, namely Δy , $2\Delta y$, $3\Delta y$, ..., where Δy , is the overall brace deformation quantity at first significant yield of the core plate. In this study the same logic of loading protocol is adopted, but instead of conducting two equal cycles, the second cycle displacement is selected larger than the previous one. For instance, instead of being Δy , Δy , $2\Delta y$, $2\Delta y$..., the new loading regime used is Δy , $1.6\Delta y$, $2\Delta y$, $2.6\Delta y$, as depicted in Figure 6.3, in order to simulate a worse seismic scenario.

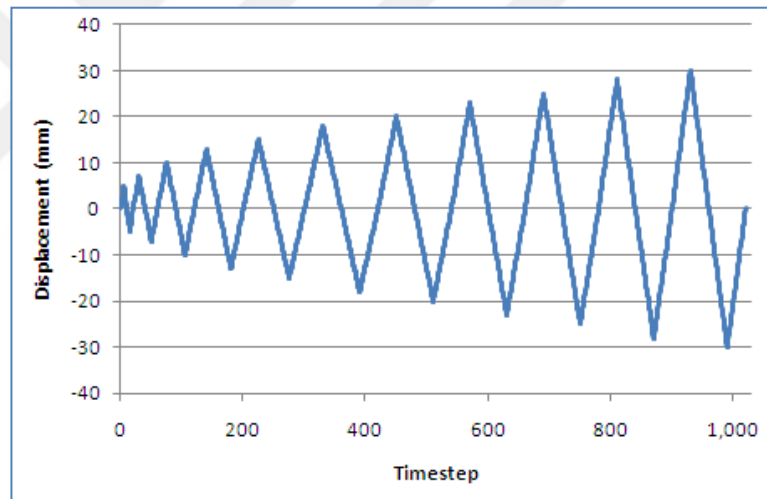


Figure 6.3 Loading protocol: Axial displacement - Timestep (1 unit=10s)

6.4 Analysis Results

From the analytical results, it is obvious that the yielding region of the specimen occurs where the curvature changes as depicted in Figure 6.4.

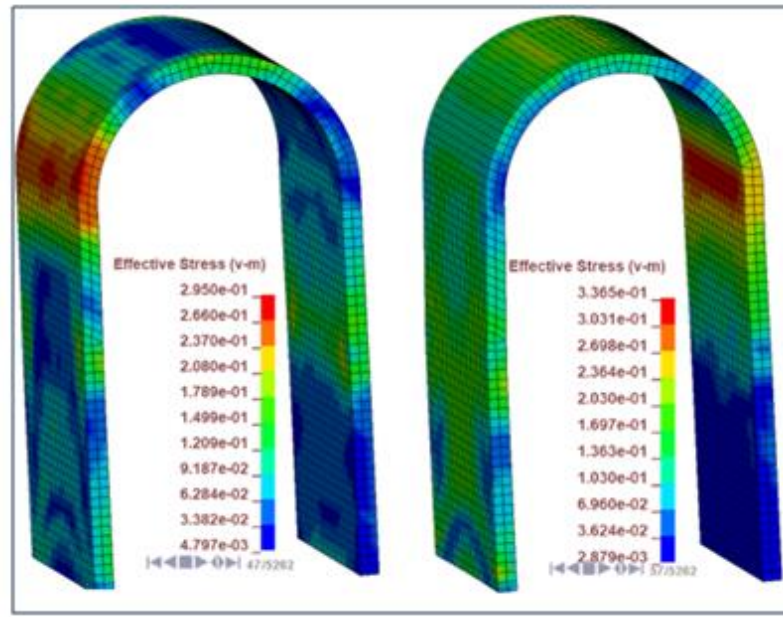


Figure 6.4 UFP effective stress v-m distribution pattern

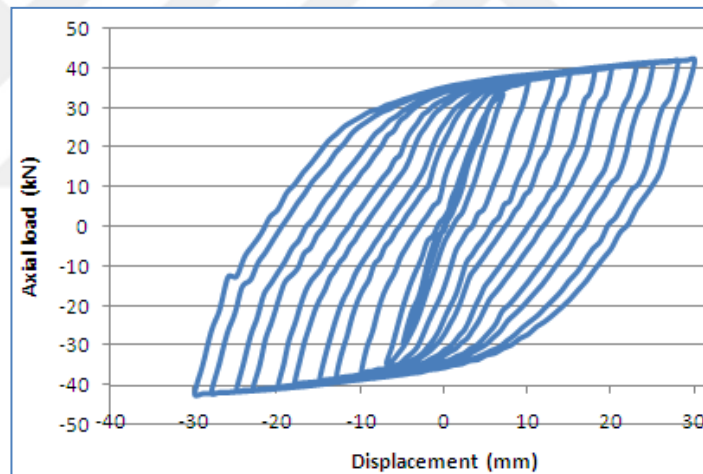


Figure 6.5 Load -Displacement curve of mini brace with 2 UFP set

Thus, the yielding point of the plate is moved back and forth along the plate during the cyclic loading regime, resulting in a redundant system with expected ample hysteresis cycles. This yielding pattern was also observed in the UFP plates experimentally tested in previous research (Baird et al., 2014). In Figure 6.4 is presented the propagation of the yield surface in time, and it is evident how in the forthcoming time step the yielding region is moved along the plate. The hysteresis produced thereof is plotted in Figure 6.5.

It is important that the initial and post-elastic stiffness, yield and ultimate force of the UFP unit, are precisely quantified, so that there is certainty that the system will be activated during the seismic event. The system yield displacement and load are respectively $\Delta y=6.1\text{mm}$ and $F_y=27.2\text{kN}$. From the analysis results the ratio of ultimate to yield load is 147%, which is within the range suggested by Kelly et al. (1972), namely 145-215%.

6.5 Low Cycle Fatigue

In order to validate the performance of the UFP unit during a seismic event, it is subjected to an extra loading of 15 cycles with the same displacement amplitude $\Delta y=30\text{mm}$, as shown in Figure 6.6.

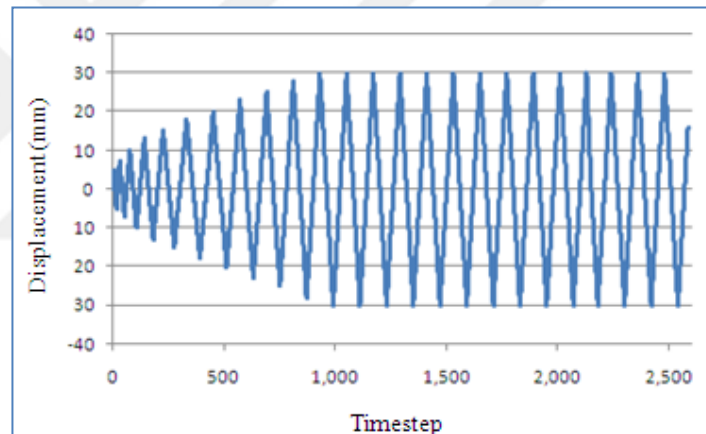


Figure 6.6 Loading protocol: Axial displacement - Timestep (1 unit=10s)

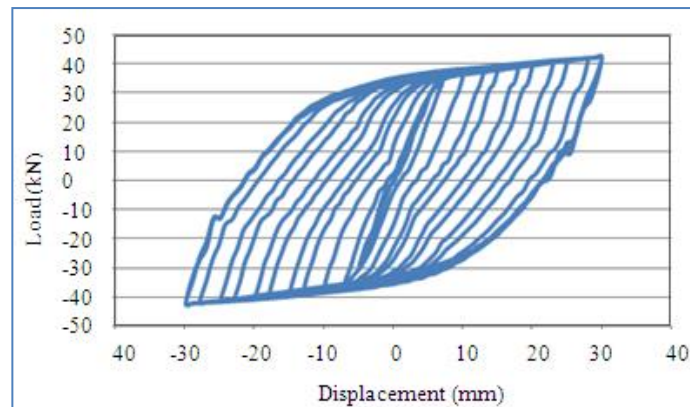


Figure 6.7 Load - Displacement curve of mini brace with 2 UFP set

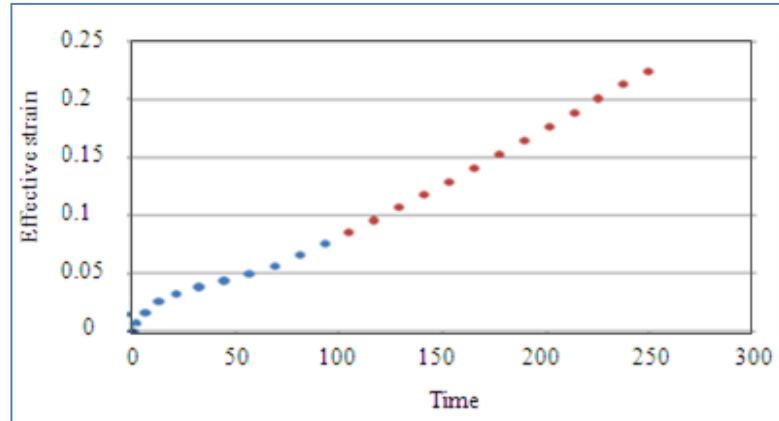


Figure 6.8 UFP effective (v-m) strain vs timestep (1unit =10s)

The analysis results show that at the end of loading (Figure 6.7), there is no obvious decrease in strength of the element. The maximum effective v-m strain generated in the element whenever the UFP unit was maximally displaced in the positive direction is depicted in Figure 6.8.

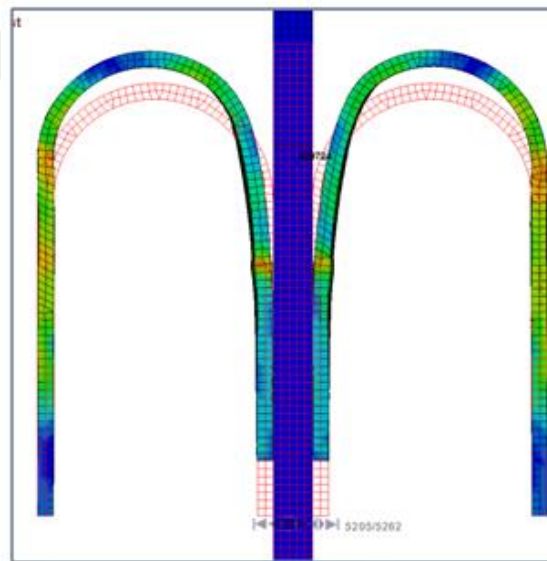


Figure 6.9 UFP deformation in the final state

Referring to the strain evolution graph, the effective strain before starting the 15 cycles of loading is 7.6 %, whereas the ultimate strain is 22%. The strain increases in a more rapid way once the 15 loading cycles start. However, the maximum strain values plotted in Figure 6.8 are within the limits of the material ultimate strain suggested by previous research (Jia & Kuwamura, 2013). The ultimate UFP plate

deformation is depicted in Figure 6.9. In this step it can be provided a comparison with the previously tested core plate as energy dissipating mechanism in the self-centering braces.

Zhou et al. (2015) in their work reported that at the load displacement of 28mm the core plates showed fractures near the notch at the inner tube welding length. Multiple-wave buckling occurred during compression, hence friction arose between the core plates and the tubes. Whereas in the UFP system, for the same level of displacement $d=28\text{mm}$, there are no fractures in the plate and there is no strength degradation of the system. Moreover, in an experimental research the UFP specimen showed no strength degradation up to approximately 80mm axial displacement, which proves that the system is far more redundant and ductile than the core plate.

6.6 Strain Rate Sensitivity of UFP Plate

The target of this study was to analytically evaluate the performance of UFP units as possible substitutes for friction and long steel core plates used as energy dissipaters in self centering braces. In this perspective, the loading rate choice was determined from the ranges used in previous experimental testing of these braces (Chou & Chung, 2014; Chou & Chen, 2015; Chou, Chung & Cheng, 2016; Chou, Tsai & Chung, 2016; Ghowsi & Sahoo, 2017) and UFP units (Baird et al., 2014), namely $0.25\text{-}0.8\text{mm/s}$ and $0.5\text{-}5\text{mm/s}$. Referring to these ranges the loading rate in this analysis was chosen $0.6\text{-}1.5\text{mm/s}$.

Strain rate sensitivity is relevant, however an analytical approach alone to evaluate its effect in this unit is not enough. The original Cowper-Symonds (CS) equation application, in order to accurately predict stresses for a broad range of strains, has undergone some modifications, but the new coefficients are chosen based on some experimental data (Alves, 2000), which are not available in this stage. On the other hand, in previous studies (Chang & Lee, 1987) it was concluded that relatively more significant strain rate sensitivity of the material occurs for monotonic

loading and less significant strain rate sensitivity for cyclic loading, which is the current loading case.

However, an analytical comparison of two strain rates and their respective effect in the maximal stresses arising in the element are presented in this section. It's worth recalling here that the full range of strain rate that might be experienced by steel building frames during earthquake ground motions falls between 10^{-6} - 10^{-1} /sec (Chang & Lee, 1987). The strain rate in the range of 10^{-6} s^{-1} to 10^{-4} s^{-1} is quasi static, whereas loading from an earthquake may impart strain rates of 10^{-3} s^{-1} to 10^{-1} s^{-1} (Ngo, Mendis, Gupta & Ramsay, 2007). Generally, an increase in flow stress is generated from an increase in strain rate, hence a higher strain rate is tested here to check its effect on stresses development. The original loading rate used in this study generates strain rates of 10^{-4} s^{-1} , hence one step higher strain rate is chosen, namely 10^{-3} s^{-1} , as for strain rates of 10^{-2} s^{-1} and higher the effect of adiabatic heating should be accounted for in the constitutive modeling (Walker, 2012). Analysis results for the stress and strain distribution in the UFP unit under two different strain rates are summarized in the following Table 6.2, whereas effective stress on the UFP unit when subjected to the original strain rate vs time is presented in Figure 6.10.

Table 6.2 Effect of strain rate on stress-strain distribution in UFP unit

Step	Stress(GPA)	Strain	Strain rate	Stress(GPA)	Strain	Strain rate	σ diff.
4	0.2811	1.66E-03	6.10E-04	0.3037	1.44E-02	-1.00E-03	8.04
5	0.313	2.78E-03	1.12E-03	0.3076	1.49E-02	5.00E-03	-1.71
30	0.3247	7.25E-03	1.79E-04	0.3163	2.80E-02	5.24E-03	-2.59
75	0.3447	1.65E-02	2.06E-04	0.3416	2.51E-02	-6.44E-04	-0.899
140	0.3609	2.61E-02	1.48E-04	0.3652	3.45E-02	1.45E-03	1.19
225	0.3661	3.27E-02	7.76E-05	0.3591	3.19E-02	-3.06E-04	-1.91
330	0.37	3.86E-02	5.62E-05	0.3646	3.70E-02	4.86E-04	-1.46
450	0.3738	4.40E-02	4.50E-05	0.3705	4.13E-02	3.58E-04	-0.883
570	0.3793	4.99E-02	4.92E-05	0.3755	4.62E-02	4.08E-04	-1.00
690	0.3824	5.68E-02	5.75E-05	0.3777	5.18E-02	4.67E-04	-1.23
810	0.3885	6.60E-02	7.67E-05	0.3822	5.63E-02	3.75E-04	-1.62
930	0.3931	7.59E-02	8.25E-05	0.3853	6.56E-02	7.75E-04	-1.98
1050	0.3961	8.54E-02	7.92E-05	0.3874	7.51E-02	7.92E-04	-2.20

Table 6.2 continues

Step	Stress(GPa)	Strain	Strain rate	Stress(GPa)	Strain	Strain rate	σ diff.
1170	0.3988	9.57E-02	8.58E-05	0.3876	8.50E-02	8.25E-04	-2.81
1290	0.401	1.07E-01	9.42E-05	0.3871	9.60E-02	9.17E-04	-3.47
1410	0.4031	1.18E-01	9.17E-05	0.3864	1.08E-01	1.00E-03	-4.14
1530	0.404	1.29E-01	9.17E-05	0.3849	1.20E-01	1.00E-03	-4.73
1650	0.4056	1.41E-01	1.00E-04	0.3839	1.32E-01	1.00E-03	-5.35
1770	0.4065	1.53E-01	1.02E-04	0.3836	1.45E-01	1.07E-03	-5.63
1890	0.4074	1.65E-01	9.92E-05	0.3835	1.58E-01	1.08E-03	-5.87
2010	0.4089	1.77E-01	1.01E-04	0.3835	1.71E-01	1.07E-03	-6.21
2130	0.4106	1.89E-01	1.01E-04	0.3836	1.83E-01	1.08E-03	-6.58
2240	0.4123	2.02E-01	1.14E-04	0.3853	1.94E-01	9.73E-04	-6.55
2360	0.4136	2.14E-01	1.01E-04	0.3851	2.08E-01	1.17E-03	-6.89
2480	0.4147	2.25E-01	9.25E-05	0.3851	2.21E-01	1.10E-03	-7.14

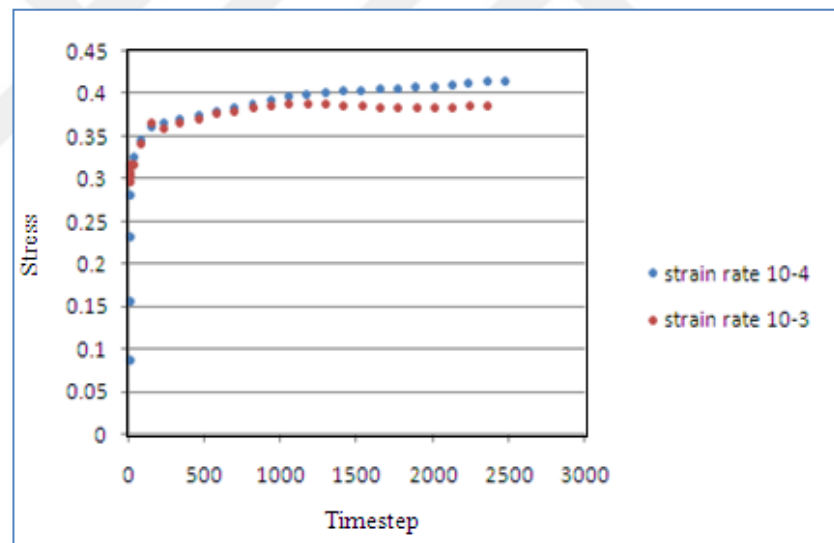


Figure 6.10 Stress (GPa) - timestep variation based on strain rate

The FEM analytical results (Figure 6.10) of this section suggest that just after the yielding point a higher stress (approx 8%) is present in the UFP unit when a higher strain rate (10 times higher) is applied. Previous studies have also concluded that mild structural steels such as A36 are characterized by the yield stresses typically only about 7% higher under seismic loading rates (Di Sarno, Elnashai & Nethercot, 2002).

Moreover, FEM analytical results indicated that there was an initial increase in stress with stress rate, but the increase decreased with increasing strain. This behavior was previously observed in experimental testing of low yield steels and was more pronounced in the steel specimen with the lowest yield strength (Saeki, Sugisawa, Yamaguchi & Wada, 1998).

It can be concluded that for a strain rate 10 times higher than the original rate, a slight initial increase in stress is observed. In a further study, evaluating the UFP behavior in a broader range of strain rates and calibrating the model parameters with experimental data can provide a relevant database for the seismic performance of this unit. However, Dusicka, Itani & Buckle (2007) performed tests on five different grades of steel and from the results it was concluded that the strain rate had little effect on fatigue life, hysteretic behavior and peak tensile stress. This shows that the results obtained in this study, despite being in a narrow range of strain rate can be quite representative of the real behavior of the specimen.

6.7 Normalizing Effect in UFP Performance

Annealing is a method of heat treatment used to alter the physical properties and at times the chemical properties of the material when exposed to heat. In the annealing process, the material is first heated beyond its critical point/recrystallization temperature and is being held at this temperature for a while before cooling it (LSTC, 2007).. Normalizing is considered a type of annealing for ferrous alloys. Hence in this section a group of materials “Steel Carbon A36” normalized in different temperatures are analyzed. The loading of the specimens is force controlled. The main aim is to see the differences in the respective hysteretic behavior of each specimen. The material type chosen in LS DYNA software is *Mat 24 without failure*, as the designers should control the failure point, by choosing the ultimate strain limit. The material type used is Type 24-MAT_PIECEWISE_LINEAR_PLASTICITY. The general system of units is respectively chosen as the GM Units (kg, mm, ms, KN, GPa, KN-mm). The materials properties are presented in Table 6.3, whereas the respective effective

stress vs strain graph for each material are depicted in Figure 6.11, Figure 6.12 and Figure 6.13 as follows:

Table 6.3 A36 normalized material properties

Material Norm @ 1650F	R_o (Kg/mm ³)	e (GPa)	Pr	σ_y (GPa)
Temp=80F	7.8334D-06	2.0684D+02	0.290	2.3934D-01
Temp=300F	7.8334D-06	2.0684D+02	0.290	1.9893D-01
Temp=500F	7.8334D-06	2.0684D+02	0.290	1.6731D-01

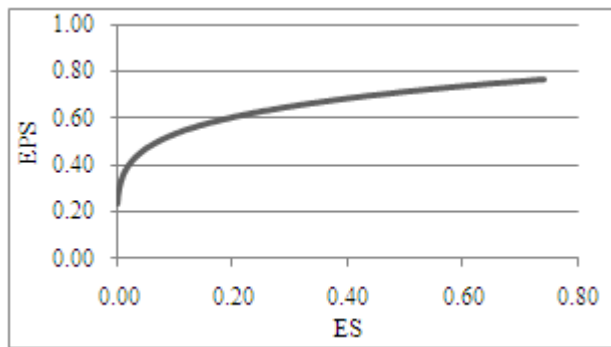


Figure 6.11 EPS vs ES Steel carbon A36Norm.@1650 Temp=80F

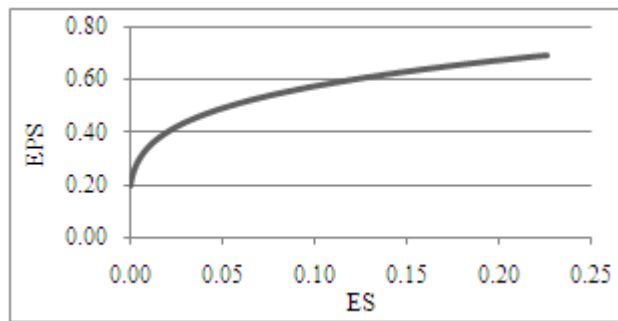


Figure 6.12 EPS vs ES Steel carbon A36Norm.@1650 Temp=300F

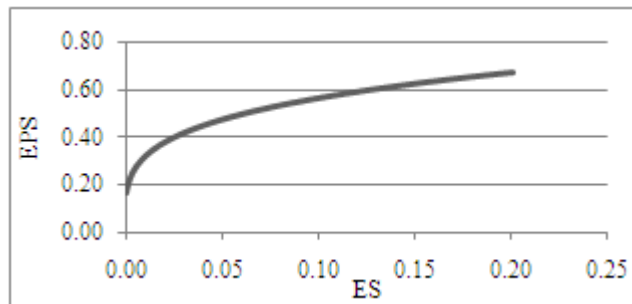


Figure 6.13 EPS vs ES Steel carbon A36Norm.@1650 Temp=500F

Table 6.4 Ultimate load and displacement for normalized A36 materials

Material <i>Steel carbon A36</i>	Ultimate displacement (mm)	Ultimate force (kN)
<i>Normalized at Temp=80F</i>	71	28
<i>Normalized at Temp=300F</i>	77	24
<i>Normalized at Temp=500F</i>	87	22

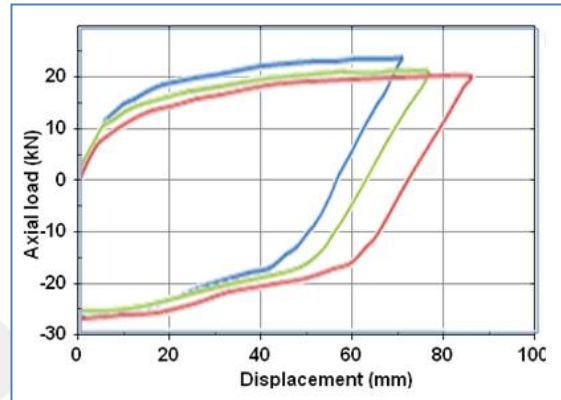


Figure 6.14 Load-displacement for different specimens – Blue (80F), Green (300F), Red (500F)

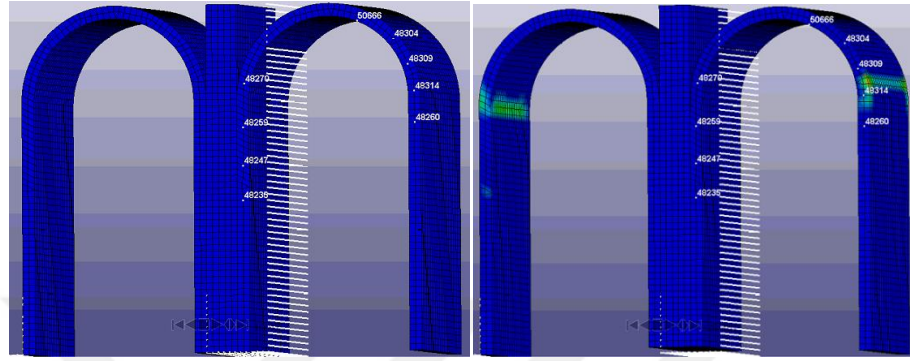
From the analysis (Figure 6.14, Table 6.4), results the following outcomes can be highlighted:

- With increasing temperature, the yielding points and the ultimate force decreases;
- When subjected to the same force level, these specimens with increasing temperature undergo larger ultimate displacements;
- With increasing temperature under the same force action, the energy dissipation increases;
- The initial and post elastic stiffness of all specimens is the same;

6.7.1 Strain Pattern in UFP Plate

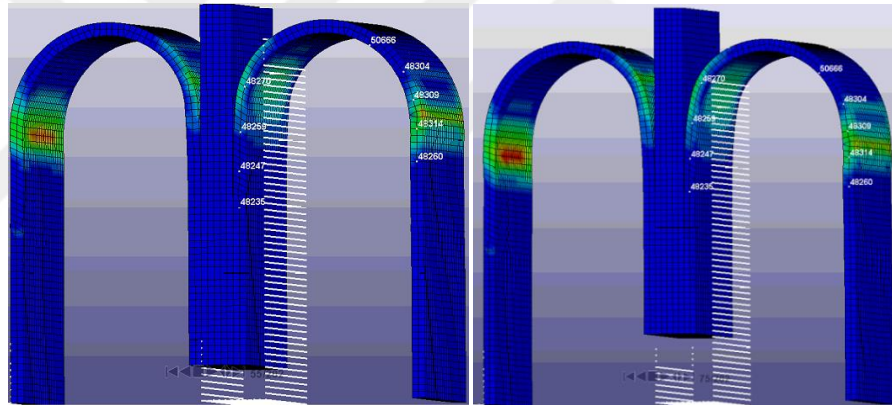
In the current section, results obtained from only one full cycle of loading up to the maximal displacement (referring to the experimental target displacement, Baird et al., 2014) are presented (Figure 6.15). The maximum strain starts developing in the region where the radius of curvature is changing at stage 28. The area of maximum

strain increases at stage 55, and then starts shifting at stage 75. This follows the logic of “strain concentrates where the curvature radius changes”. This behavior is present in both legs of the UFP. The nodes highlighted in white define the area where the maximum strain is developed throughout a full loading cycle.



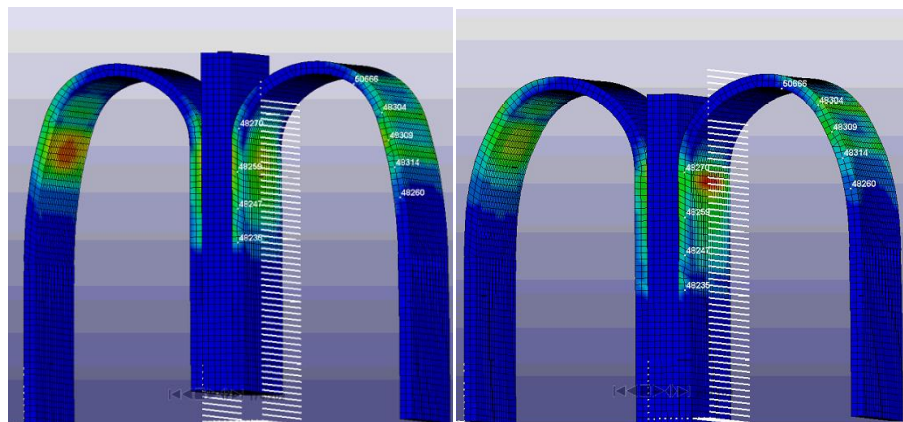
(a) Stage 0

(b) Stage 28



(b) Stage 55

(d) Stage 75



(a) Stage 174

(d) Stage 201

Figure 6.15 Effective plastic strain development in time

When the UFP starts the inverse loading regime (*Stage 174*), the maximum strain region continues to shift, which is the main characteristic that makes this system redundant when compared to other direct tension/compression systems, in which yielding is localized in one single region.

6.8 UFP Additive Behavior

In the previous sections it was noted that the designer can choose an appropriate UFP size and the material matching his needs, in order to reach the load and displacement capacity he necessitates. However, in this section, there one main issue that needs to be discussed: *“Is the hysteresis of one UFP set enough for the designer, in terms of the data provided, so that he can design a multiple set of UFPs based on these data?”* Hence a straightforward comparison between the hysteretic behavior of one single UFP set and the double UFP set is presented in this section. Following a 10mm displacement step, the axial forces arising in the mini-braces are recorded and presented in Table 6.5 and Table 6.6.

Table 6.5 Ratio of forces in both specimens with increasing displacement

Displacement (mm)	F1(kN) 1se tUFP	F2(kN) 2set UFP	F2/F1
9.98	13	23.3	1.7923
19.3	16.3	30.9	1.8957
29.4	18.1	34.8	1.9226
39.5	19.4	37.6	1.9381
49.5	20.1	39.5	1.9651
59.2	20.6	40.8	1.9805

Table 6.6 Ratio of forces in both specimens with decreasing displacement

Displacement (mm)	F1(kN) 1set UFP	F2(kN) 2set UFP	F2/F1
50.8	-8.01	-14.7	1.8352
40.7	-15.5	-25.6	1.6516
30.5	-17.2	-32.1	1.8662
20.6	-20.2	-38.6	1.9108
10.8	-23.1	-43.7	1.8917
0.619	-23.5	-45.5	1.9361

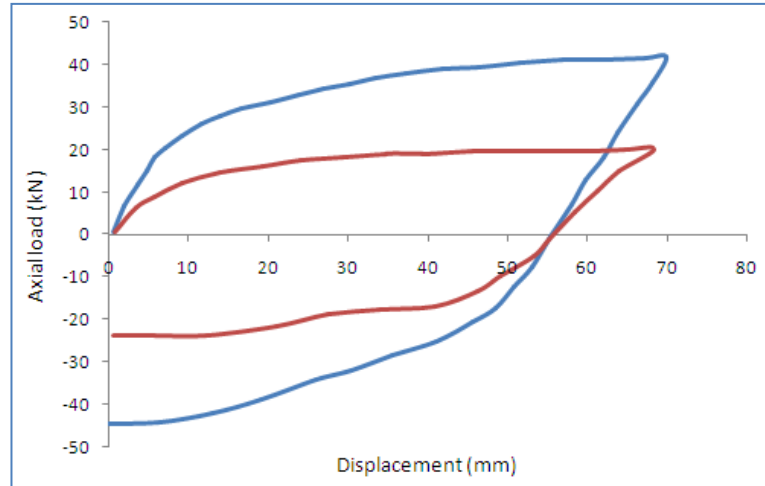


Figure 6.16 Hysteresis of 1-UFP (red) & 2-UFP (blue) set Carbon Steel A36 Annealed Temp=80F

From the data available in Table 6.5 and Table 6.6, it can be concluded that at the same displacement of the UFPs the force generated in the double UFP set is twice the force generated in the single UFP set. This behavior, as expected, is present in both situations where displacement is increasing and decreasing. Both hysteresis are overlapped in order to graphically observe this difference (Figure 6.16).

6.9 Full Brace with UFP

The specimens discussed so far represent individual parts, which show the accurate behavior of the UFP, but none of them can be implemented alone as a large-scale energy dissipation brace (Appendix F). Their maximum load capacity was only approximately 40kN. Hence in the following section, it is presented a multiple UFP brace. It is composed of 10 sets of UFPs, and reaches up to 200kN ultimate load. The brace is presented in Figure 6.17.

The material chosen for the UFP plates, is the same of the previous specimens in this section, namely Steel Carbon A36 Annealed Temp=80F. In order to analyze the additive performance, the same comparison of parameters is done as in the previous section. The displacement interval in which the graph points are checked is 10mm.

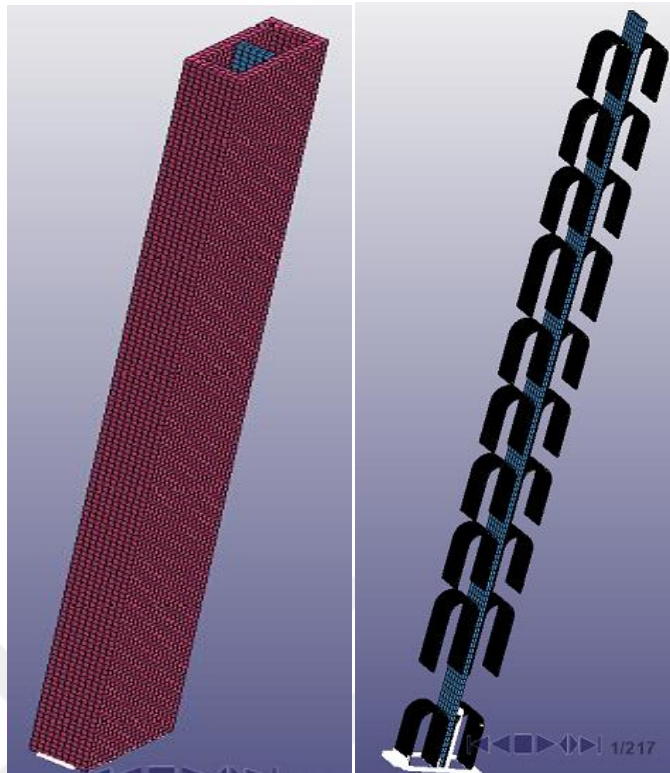


Figure 6.17 Full brace with 10 UFP sets

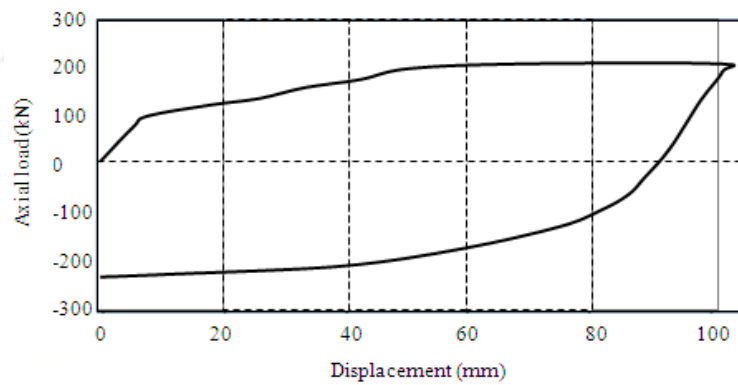


Figure 6.18 Load (kN) – displacement (mm) of 10 set UFP brace

Table 6.7 Ratio of forces generated in 1 set UFP and 10 set UFP brace

Displacement (mm)	F1 (kN) 1 set UFP	F2(kN) 10 set UFP	F2/F1
9.98	13	107	8.23
19.3	16.3	133	8.15
29.4	18.1	152	8.39
39.5	19.4	176	9.07
49.5	20.1	194	9.65
59.2	20.6	206	10

The data provided (Table 6.7) from the presented hysteresis of the large specimen, and the comparison to the single UFP specimen, show that at small displacements the force generated in all the UFPs is ~8.2 times larger than in one single UFP. Whereas closer to the ultimate displacements this ratio becomes approximately 10. In the previous comparison the ratio of forces was nearly constant though time, hence for these intermediate values that show some divergence from the linear ratio of forces, experimental testing has to be performed. From all the analysis performed it can be concluded that the ultimate load of the UFP is proportionally linear to the number of UFP sets in a brace. Hence a brace with 2 UFP sets has an ultimate load capacity equal to twice the one of one UFP set. As well a brace with 10 UFP sets has an ultimate load capacity equal to ten times the one of one UFP set.

Another important parameter necessary for the determination of the additive behavior, is the initial stiffness of the brace specimens. In Table 6.8 the initial stiffnesses of all specimens are presented. The points on their right are respectively the points in each graph, obtained in order to calculate the initial stiffness.

Table 6.8 Initial stiffness of all specimens

Specimen (UFP set nr)	Initial stiffness (kN/mm)	Curve point (x;y) checked
15	26.2	(3.08;80.7)
10	16.49	(4.73; 77.9)
2	3.27	(4.37; 14.3)
1	1.69	(5.34; 9.0)

Initial stiffness ratio of:

$$K_{i(15 - \text{UFP})} / K_{i(10 - \text{UFP})} = 26.2\text{kN/mm}/16.49\text{kN/mm} = 1.588$$

$$K_{i(10 - \text{UFP})} / K_{i(2 - \text{UFP})} = 16.49\text{kN/mm}/ 3.27\text{kN/mm} = 5.04$$

$$K_{i(2 - \text{UFP})} / K_{i(1 - \text{UFP})} = 3.27\text{kN/mm} /1.69\text{kN/mm} = 1.9345$$

After performing the necessary ratios, it is concluded that the initial stiffness of a brace composed of a set of UFPs is linearly proportional to the initial stiffness of the single UFP set. Namely:

- The 15 – UFP set specimen has $\sim 1.5(1.588)$ times higher stiffness than 10 – UFP set specimen.
- The 10 – UFP set specimen has $\sim 5(5.04)$ times higher stiffness than 2 – UFP set specimen.
- The 2 – UFP set specimen has $\sim 2(1.9345)$ times higher stiffness than 1 – UFP set specimen.

When designing a seismic energy dissipative damper, another important parameter to be checked is the yield displacement, and the respective yielding load. The yield displacement and the respective yield load are recorded and presented in Table 6.9. The yielding point in the graph was denoted the one that represented the last step of the linear elastic behavior of the specimen. As expected, the yield displacement is the same for all specimens. On the other hand, the yield load is linearly proportional to the number of UFP sets used in the brace.

Table 6.9 Yield displacement and load for each specimen

Specimen (UFP set nr)	Yield displacement (mm)	Yield load (kN)
15	4.37	119
10	4.73	77.9
2	4.66	15.1
1	4.41	8.51

This damper provides an enhancement when compared to single member tension/compression dampers, as it has a low yielding point but a high yield ultimate load. After analyzing the data provided for all four specimens it is concluded for this section that:

- Yield displacement is the same for all specimens;
- Yielding load is linearly proportional to the number of UFP sets used in the brace;
- This damper provides advantages over other dampers because it has a low yielding point, and high ultimate strength and displacement.

6.10 Enhanced UFP Brace - Self-Centering

Seismic design philosophy has evolved to the level that modern structural systems can save their integrity and occupants lives even after strong earthquakes. This is mainly possible due to the distribution of the inelastic action in some well detailed regions of structural members, which limits the seismic forces induced to the rest of the system. However, the deformations remaining in the structure after significant seismic events can reduce the effective ductility capacity to resist future earthquakes according to (Maffei, Telleen & Nakayama, 2008) or even lead to progressive collapse. Previous research has shown that buildings with residual drift ratios greater than 1% would not be safe (Erochko, Christopoulos, Tremblay & Choi, 2010). Moreover, it is essential for modern seismic design to obtain resilient structures, that require minimal or no repairs after a seismic event. Therefore, structural systems that are forced back to their initial position after an earthquake, generally known as “self-centering systems”, have recently gained priority in the structural engineering research. The main self-centering systems developed so far are rocking walls (Priestley, Sritharan, Conley & Pampanin, 1999; Wada et al., 2010; Qu, Wada, Motoyui, Sakata & Kishiki, 2012), self-centering moment frames (Ricles, Sause, Garlock & Zhao, 2001; Rojas, Ricles & Sause, 2005; Kim & Christopoulos, 2008; Lin, Sause & Ricles, 2013) and self-centering braces. The latter provide the optimal solution to obtain a resilient structure, as their implementation does not require major modifications to the other parts of the structural system. At the current stage of development, these braces can accommodate up to 3.9% drift in the test frame and have high axial capacity, comparable to available buckling restrained braces (Erochko, Christopoulos & Tremblay, 2014a, 2014b).

However, there is still need for improvement of the energy dissipating mechanisms incorporated in these braces. Researchers focus so far has been mostly oriented towards the friction based (Christopoulos, Tremblay & Kim, 2008; Erochko et al., 2010, 2013, 2014a, 2014b; Zhu & Zhang, 2007, 2008), and yield based energy dissipation systems (Chou et al., 2014, 2015, 2016a, 2016b; Zhou et al., 2015; Xhahysa, Kahraman & Misir, 2017; Mashal, Palermo & Keats, 2019).

In recent works alternative energy dissipation systems have emerged, such as fluidic self-centering braces (Kitayama & Constantinou, 2016a, 2016b, 2017), shape memory alloyed wires (Miller, Fahnestock & Eartherton, 2012), piston based self-centering braces (Haque & Alam, 2017) and magnetorheological fluid braces (Xu, Xie & Li, 2018). As these recent systems are less favorable for mass production of the braces and can possibly lead to higher costs, the optimal energy dissipation mechanisms can be considered the friction based and yield based mechanisms.

However, in friction-based systems the eccentricity of friction bolts with respect to the tube surface causes significant flexing. Moreover, there is susceptibility to bolt relaxation and long-term creep of friction material, and additionally high velocity change of stiffness was the main cause for considerable peak floor accelerations (Erochko et al., 2013). Whereas in the braces where the energy dissipation was obtained from a steel plate core yielding in tension and compression, residual deformation and local buckling were observed (Zhou et al., 2015). Dissipaters using direct shear, tension, compression mechanisms are less suitable, as they tend to generate large strain demands. Whereas, flexural yielding uses more indirect load paths, lowering the strain demand, hence resulting in a larger displacement capacity (Henry, Aaleti, Sritharan & Ingham, 2009).

Hence, in this section, an alternative energy dissipating device that can be used as the primary or secondary fuse in self-centering braces is proposed. A set of U-shape flexural plates (UFP) that yield in flexure, provide a low-cost system with a wide and stable hysteresis. Once incorporated in the self-centering braces this mechanism is expected to show a gradual change in stiffness, which is the main factor to avoid peak floor accelerations observed in friction-based systems (Wiebe & Christopoulos 2010, 2011). On the other hand, if compared to previous yielding-based energy dissipating systems, the mobile yielding region within the UFP makes the mechanism more redundant. The proposed solution has been studied by conducting finite element modeling.

6.10.1 Dual Tube Concept

Self-centering braces have evolved in terms of the material used and techniques of construction, but the working mechanism of the system has remained the same. The schematic representation of the original self-centering brace is shown in Figure 6.19b.

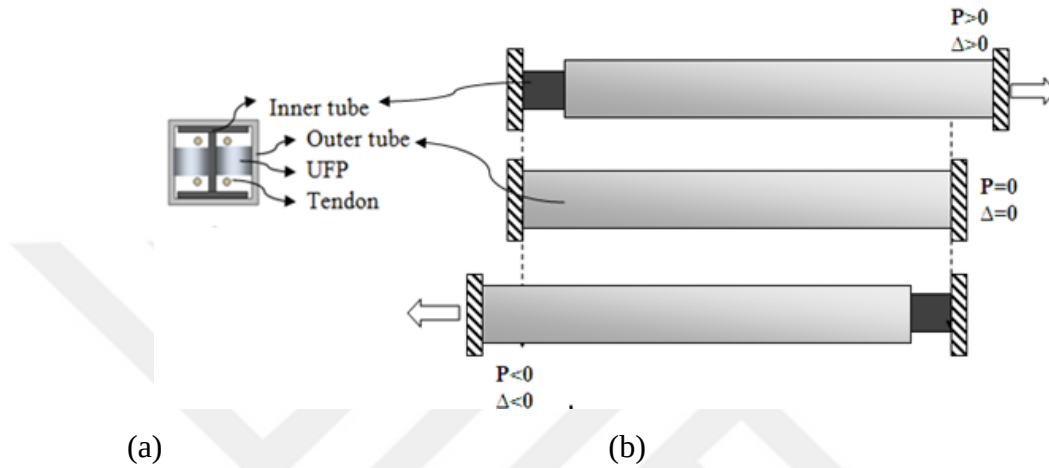


Figure 6.19 (a) Brace section with U-shape flexural plate energy dissipation (b) Double tube in motion

The outer and the inner tube move axially relative to each other when subjected to tensile or compressive loads. The prestressed tendons anchored in the end plates bring the system back to initial position with zero residual drift. Different energy dissipating mechanisms can be placed between these tubes. In this study a set of UFPs placed longitudinally between the tubes that act as the primary energy dissipating mechanism is analyzed. Its schematic representation is shown in Figure 6.19a, whereas the detailed brace configuration is depicted in Figure 6.20.

As in the previous self-centering braces, the outer and inner tube serve for pushing the tendons and restraining the energy dissipation mechanism, in order to have a predefined hysteresis. This working mechanism avoids the usage of a third member, which was previously needed in yield-based self-centering braces (Chou et al., 2016a, 2016b) to restrain the core plate. Therefore, it reduces the brace complexity in design and application, and consequently reduces its weight and cost.

In this brace basalt fiber-reinforced polymer BFRP tendons are chosen instead of the metallic strands used in previous versions as the prestressing system, due to their lower elastic modulus and a higher elastic elongation than a prestressed metallic strand, and lower cost than other fiber strand tendons. Moreover, previous research has shown that the cyclic elastic modulus of the aramid fiber is 1.5–2 times larger than monotonic loading (Christopoulos et al., 2008). Moreover, the ratios of the ultimate elastic modulus to the initial elastic for BFRP tendons vary only 1.02-1.12 under cyclic loading (Zhou et al., 2015).

In Figure 6.20 are presented the structural details of the UFP dual tube SC Brace modeled with LS DYNA, with its main sub-systems, respectively 10 sets of UFPs as the energy dissipation mechanism and the prestressing system.

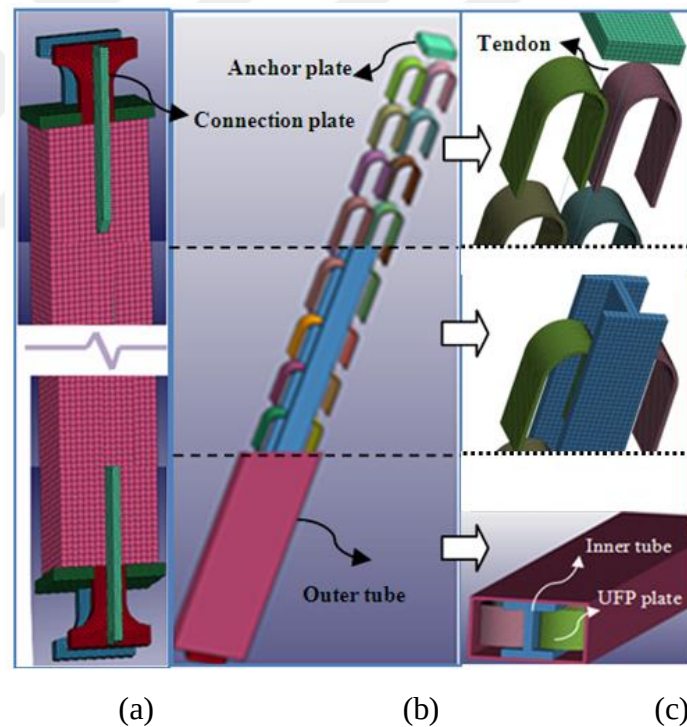


Figure 6.20 (a) Full brace (b) Partial decomposition of brace in 3 steps (c) Representative brace slices

Full brace is depicted in Figure 6.20a, where only the outer tube, anchor plate and the connection plates are visible. In order to get a better view of each of the elements composing the brace, a partial 3-step brace decomposition is presented in Figure 6.20b.

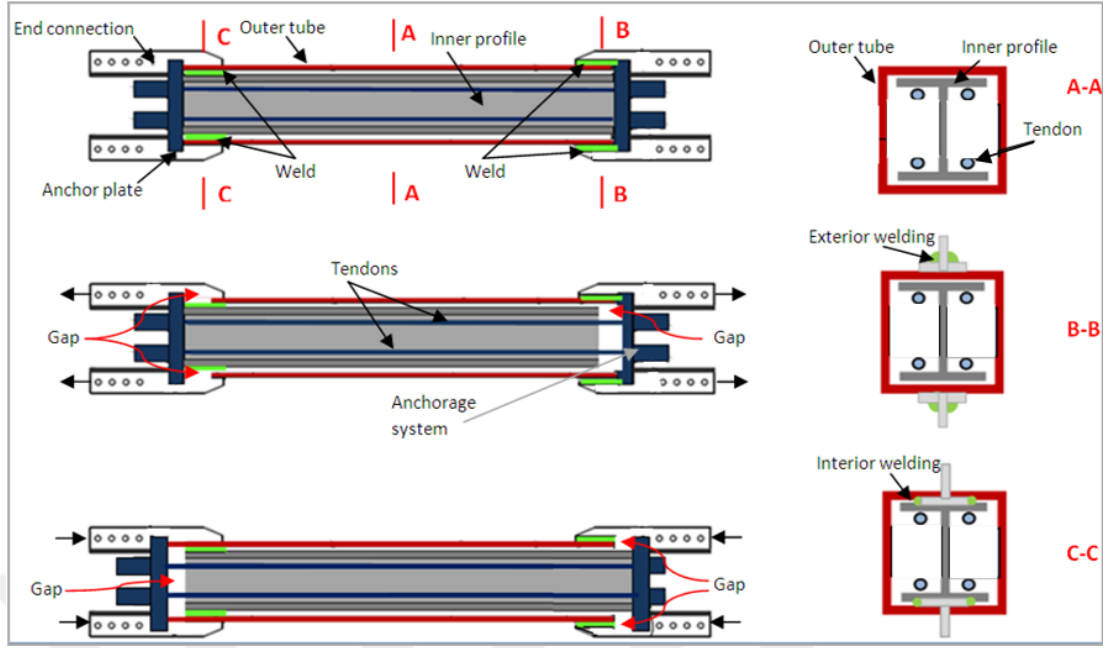
The outer tube is removed first in order to visualize the inner tube and the UFP units, whereas in the last step the inner tube is removed as well so that the tendons anchored in the anchor plate can become visible.

The UFP unit implemented in this brace has the same dimensions and material properties as the prototype experimentally tested in a previous research (Baird et al., 2014). One side of the UFP is bolted to the inner tube, whereas the other side to the outer tube at 100mm from the UFP baseline. The end plates where the tendons are anchored, can move freely, as they are not connected to any of the brace members.

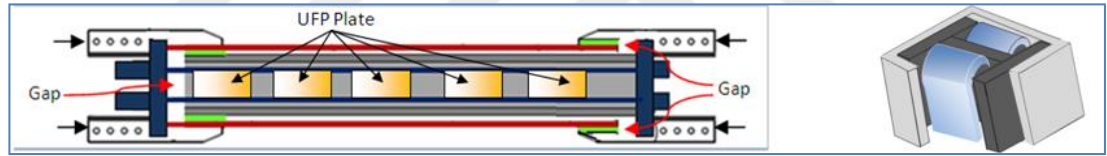
When the brace is subjected to tensile forces, the inner tube moves leftward and the outer tube rightward, together with the UFP legs attached to each of them. This telescope like motion of the tubes activates flexure in the UFP units, which provide the energy dissipation mechanism in this system. Tendons remain always in tension with the prestressing force that is responsible for the self-centering of the system. The compressive behavior of the brace follows the same logic of the tensile state. This flexural-based yielding is different from previous yield based self-centering braces, where compression or tension was activated in the yielding core plate depending on the overall brace loading regime. This makes the new brace hysteresis more favorable to a symmetric behavior.

6.10.2 Brace Mechanics

Based on the working mechanism of the brace (Figure 6.21), it is observed that the UFP set and the dual tube system work in parallel. Hence, the total hysteresis curve can be obtained by directly adding the hysteresis response curves of the two systems together, as shown in Figure 6.22. The hysteretic curve of the prestressed dual tube system is presented in Figure 6.22a, whereas in Figure 9b is depicted the hysteretic curve of the UFP set. The final curve of the two systems merged is presented in Figure 6.22c.



(a)



(b)

Figure 6.21 Schematic representation of brace (a) without UFPs (b) with UFPs

F_0 is the prestress force in the BFRP tendons; k_{e1} is the initial stiffness of the self-centering system, including the stiffness of the inner tube k_i , outer tube k_o , and BFRP tendons k_t ; Δ_0 is the activation displacement of the self-centering system, k_{e2} is the elastic stiffness of the UFP set; k_{p2} is post-yield stiffness of the UFP set; k_{pe} is post-activation stiffness of the system; F_{ty} and Δ_{ty} are the yield force and displacement of the UFP set when the brace is in tension; F_{cy} and Δ_{cy} are the yielding force and displacement of the UFP set when the brace is in compression and Δ_r is the possible residual deformation when the brace is unloaded.

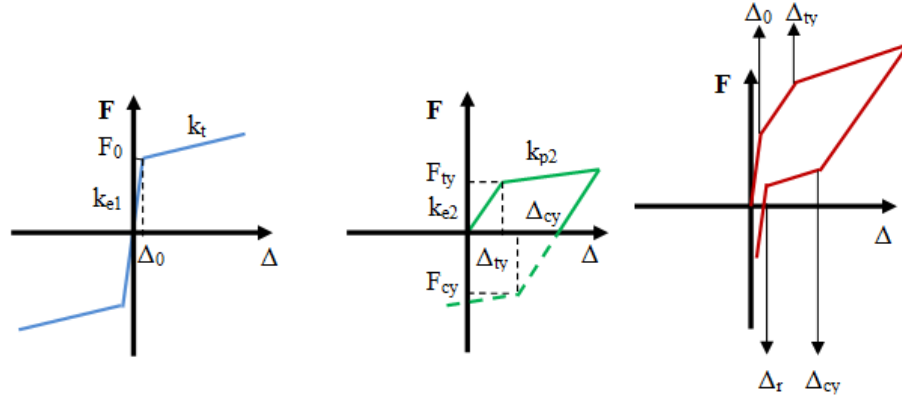


Figure 6.22 Load- displacement curve of (a) prestressed dual tube system (b) UFP set (c) Brace

When the brace is at rest, the initial pretension P applies compressive force to the inner tube and the outer tube, relative to their axial stiffness; hence the brace shows a relatively high initial stiffness k_{e1} upon loading until activation is reached. Initial stiffness cannot be defined by the axial stiffness of the brace members alone, but also the axial stiffness of the connection plates (k_{cp}) should be included in order to obtain the real initial stiffness. Hence the pre-activation stiffness of the brace theoretically can be expressed as in the following Equation 6.6:

$$k_{e1} = \frac{(k_i + k_0 + k_t)k_{cp}}{k_i + k_0 + k_t + k_{cp}} \quad (6.6)$$

This stiffness is denoted as a “theoretical value”, because in reality the tube length difference can reduce it significantly (Erochko et al., 2014a,b). After activation, the UFP plates are still in the elastic phase, and before reaching the yield displacement (Δ_{ty}), the system shows a lower stiffness than k_{e1} ; that is affected by the UFPs (k_{e2}) and tendon elastic stiffness (k_t). Beyond this point the stiffness of the system is basically equal to the tendons stiffness (k_t), as the UFPs plate post yield stiffness is relatively low. However, including again, the effect of the connection plate axial stiffness, the resulting post-elastic stiffness of the system can be expressed as Equation 6.7:

$$k_{pe} = \frac{k_t k_{cp}}{k_t + k_{cp}} \quad (6.7)$$

Upon unloading the initial stiffness of the UFP plates is activated again, until the yield displacement is reached. In case the pretension force is less than the yield force of the UFPs including the effect of hardening, residual deformation (ur) may be present in the system.

6.10.3 Prestressing of Alternative UFP-Set Energy Dissipation System

Before stepping to the prestressing of the full specimen, in the following section a small brace with only one set of UFPs is prestressed, in order to obtain the new hysteresis of the brace. It is recalled here the schematic representation of the brace, as shown in Figure 6.23.

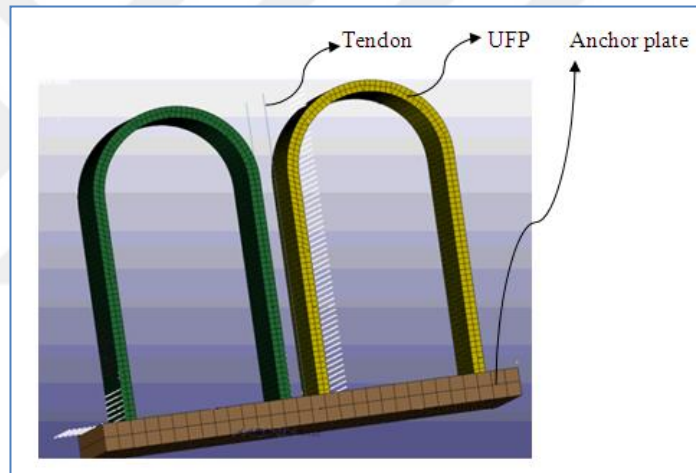


Figure 6.23 Mini-brace inner elements: UFPs, tendons and anchor plate

The elements added here are the two tendons and the plate where these tendons are anchored. The outer tube is not shown in order for the internal configuration of the brace to be clear. For the brace simulation in the LS DYNA model “Beam” section is chosen, with an element formation: equation 6, namely discrete beam cable. Volume and area are as well determined here, whereas the material type selected is MAT_CABLE_DISCRETE_BEAM. The initial stiffness of the tendons is chosen as 50GPa, a value used for BFRP tendons in experimental tests, whereas the prestressing force is chosen 10kN. It is necessary to highlight the reason behind this choice of initial prestress force.

The ultimate load of the UFP set is $\sim 20\text{kN}$, hence it is needed the same prestress force so that once the outer force will become zero, this prestress force will bring the system back to zero deformation, because it will overcome the force produced in the energy dissipating device.

As expected, when a yielding energy dissipation device with prestressing tendons in a double tube system are joined, a flag shape hysteresis is expected. This is exactly the hysteresis shown in Figure 6.24. Here it is noted that the initial stiffness is very high as expected. For the initial stiffness contribute additively the inner tube, the outer tube and the tendons, whereas after this activation force is passed, the system shows to a lower stiffness.

The post elastic stiffness of the system is equal to the stiffness of the tendons and the energy dissipating device after yield. In order to have a clear view of how the hysteresis of the brace changes after the prestressing, in Figure 6.25 the respective hysteresis are overlapped. Higher post-yield stiffness is observed in the self-centered specimen when compared to the original specimen, due to the presence of tendons. As expected, the prestressing force was enough to bring back the system to the zero deformation.

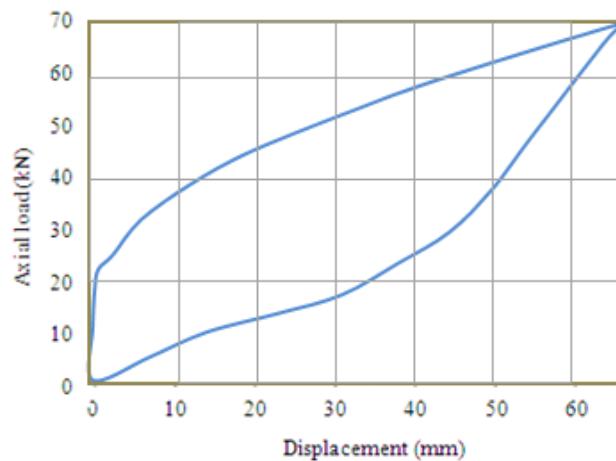


Figure 6.24 Load (kN) – Displacement (mm) of the self-centering UFP brace

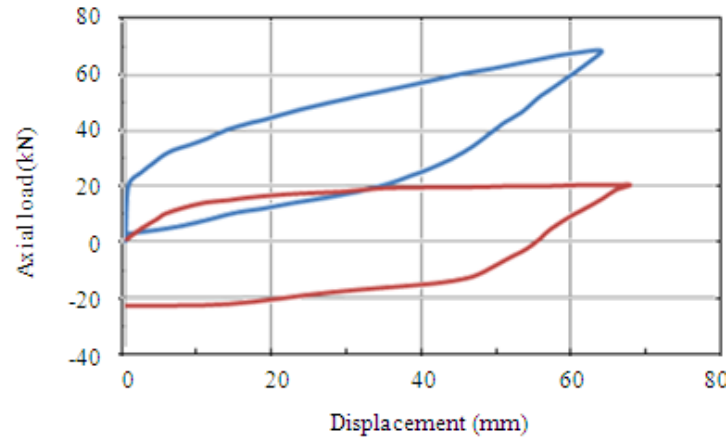


Figure 6.25 Hysteresis of the self-centering UFP brace (blue) vs the non-prestressed UFP brace (red)

Another important parameter that has to be controlled when designing energy dissipating devices is the effective plastic strain. The distribution pattern is important to define the weak parts that are prone to yield in the system, and in case this yielding region moves in the element and is not concentrated in one point, it makes the system safer, more redundant. Actually, the same distribution pattern of effective plastic strain that was present in the original non-prestressed UFP set brace, is again present after prestressing is applied. This strain concentrates in the regions where the radius of curvature changes, and moreover this yielding region shifts up and down during the UFP rolling action, making this system favorable for the low cycle fatigue tests. In graphical representation, this behavior is presented in Figure 6.26.

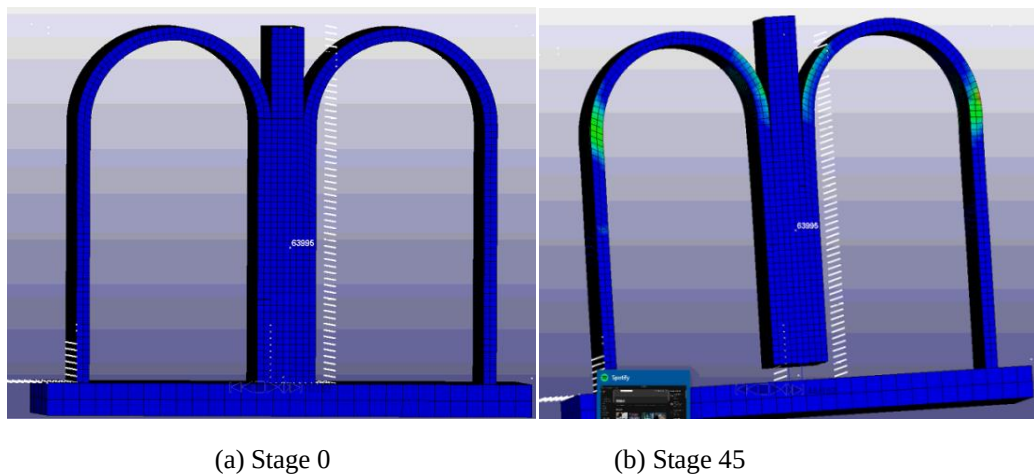
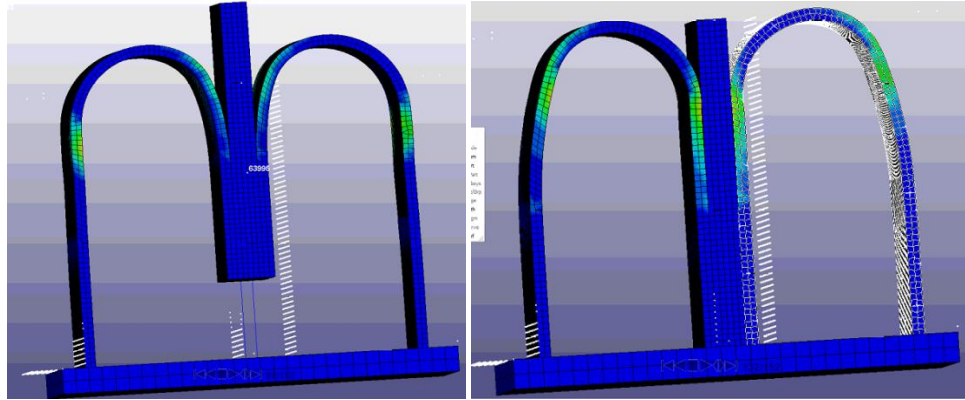


Figure 6.26 Effective plastic strain distribution in different stages



(c) Stage 88

(d) Stage 132

Figure 6.26 continues

Despite the favorable strain distribution, it is important to control also in quantitative terms, what is the maximum strain that develops in the UFP after prestressing has been applied. As shown in Figure 6.27, there is a plateau of the strain from time 9ms up to the end of cycle, when the UFP legs reach again the zero position. Strain reaches a maximum value in this plateau of ~ 0.07 , which is lower than the maximum strain observed in the original UFP with no prestressing, namely 0.08. This makes the system favorable in terms of strain development.

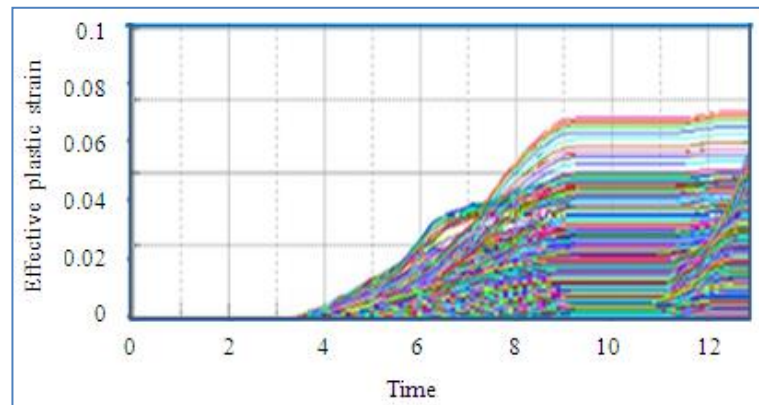


Figure 6.27 Effective plastic strain in all UFP elements vs time

The next elements that have to be controlled in this round are the tendons. Their axial force vs time and axial force vs axial displacement are shown in Figure 6.28, Figure 6.29 respectively.

As expected, the initial prestress (20kN), the final zero deformation and the linear proportional behavior of the force vs displacement of the tendons characterize their behavior.

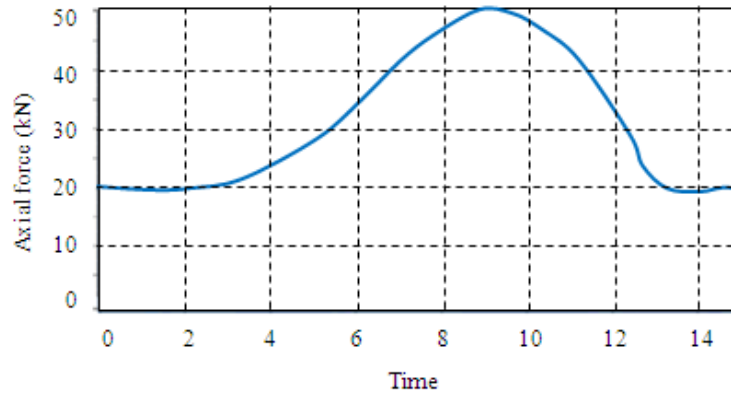


Figure 6.28 Tendon axial force (kN) vs time

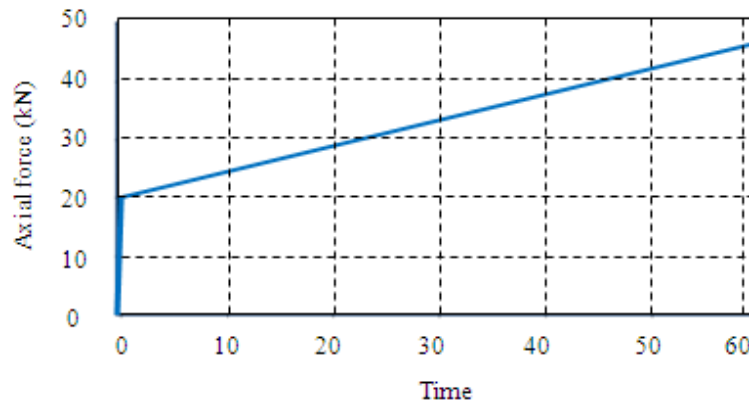


Figure 6.29 Tendon axial force (kN) vs displacement (mm)

It can be concluded that:

- The prestressed system of tendons for the UFP brace system has satisfactorily reached the flag- shape hysteresis.
- The prestress force chosen for the current case is enough to bring the system back to zero deformation after the ultimate load of the energy dissipation system has been reached.
- There is a decrease in effective plastic strain in the UFP set when compared to the original UFP set with no-pretension.
- The tendons are always in tension.

- As suggested a prediction of the overall initial elastic brace stiffness to be used in a global structural analysis should also include the deformations of the brace connections outside of the length L . The overall stiffness including end connection flexibility has been found to be in the range of 50-70% of the stiffness of the brace K_{be} , for typical brace geometries, in previous brace designs (Barbagallo et al., 2017).

6.11 Cyclic Performance of Prestressed UFP Brace

An analytical approach using LS DYNA finite element software was used to generate the cyclic behavior of a single set and multiple set of UFPs incorporated in a prestressed double tube system. Specifically, in *Specimen 1* only one set of UFPs is used, whereas in *Specimen 2*, 10 sets of UFPs. The outer tube has a rectangular hollow cross section HSS156x276x12mm. Inner tube is an H section with overall height $H=140\text{mm}$, flange width $B=120\text{mm}$ and with web and flange thickness $t=20\text{mm}$. The tubes have a total length of 2860mm for the large-scale brace, whereas only 220mm for the brace with 1 UFP set. The end plates are steel plates with dimensions 280x280x35mm.

The UFP unit dimensions and material are the same as presented in the previously performed experimental research (Baird et al., 2014), where it is suggested that the behavior of the UFP should be calibrated by altering the post-yield plastic material properties. The parameters that were found to best match the experimental results were combined cyclic hardening with a Q -infinity of 120MPa and Hardening Parameter of 4.5. Prestress in the system is provided by BFRP tendons, with elastic modulus 40GPa. Initial prestress force in the specimens 1 and 2 is chosen 34kN and 240kN, respectively. These values are larger than the ultimate force produced by the UFP sets, namely 22kN and 220kN, in order to avoid residual deformations.

For a 30° angle brace installation in an actual frame, the elongation of the brace is 0.89% in case an interstory drift of 2% is targeted. This yields a corresponding 26.7mm axial deformation of the brace. Tendons are modelled with the material type “071 Cable Discrete Beam”, and prestress is applied as an initial tensile force “ F_0 ”.

Automatic surface to surface contact between surfaces coming together after some time is applied. The database binary plot has a time step of 0.01.

6.11.1 Cyclic Performance of Specimen 1

The loading protocol is displacement controlled, with 4 cycles of increasing amplitude. Each cycle amplitude is respectively $2\Delta y$, $4\Delta y$, $6\Delta y$ and $8\Delta y$, where Δy is the overall brace deformation quantity at first significant yield of the UFP unit. The specimen hysteresis is shown in Figure 6.30. The flag shape hysteresis curve shows that the self-centering system is working properly. It is recalled here the previous experimental result (Baird et al., 2014), which showed that the ratio of maximum force generated in the UFP under negative/positive loading was ~ 1.09 . This explains the approximately symmetric behavior of the new system when loaded in tension and compression, as shown in Figure 6.30.

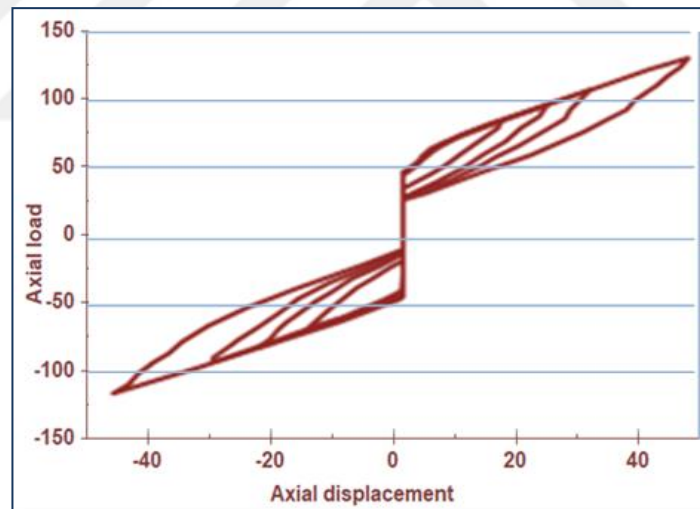


Figure 6.30 Load (kN)- displacement (mm) curve of Specimen 1

For each cycle the maximum positive and negative force in the brace are respectively (+70kN/-73kN; +82/-86kN; +92/-96kN; +114/-118kN). This behavior conforms to one of the main requirements for braces that limits this ratio up to 1.3. Previous SCED (Zhou et al., 2015) braces with core plates yielding in tension/compression showed asymmetry during tension and compression cycles, due to multi-wave buckling of core plates causing friction between the core plates and the

tubes. On the other hand, when compared to friction based SCED braces, this new system provides a more gradual stiffness change in the high velocity range. This can possibly avoid the peak floor acceleration that was observed in frames that incorporated friction based SCED braces (Erochko et al., 2013).

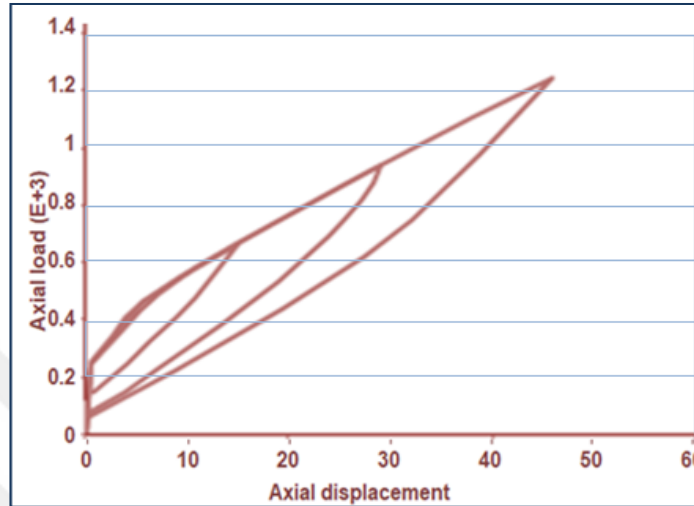


Figure 6.31 Load (kN)- displacement (mm) curve of Specimen 1

6.11.2 Cyclic Performance of Specimen 2

In this specimen 10 sets of UFPs are used in order to check the brace overall performance and the resulting flag shape hysteresis. Prestress in this system is generated by 4 Φ 14 BFRP tendons. The loading protocol is displacement controlled, with 3 cycles (only with positive displacement) of increasing amplitude. Each cycle amplitude is respectively 3 Δy , 6 Δy and 9 Δy , where Δy is the overall brace deformation quantity at first significant yield of the UFP unit. The specimen hysteresis is shown in Figure 6.31. The flag shape hysteresis curve shows again that the self-centering system is working properly. The brace maximum load for 45mm axial elongation was 1245kN, however higher load capacities can be reached with wider UFP plates and stiffer tendons. Assuming that the brace would be installed at a 30° (Zhou et al., 2015) angle in an actual frame, for an inter-story displacement of 2%, the elongation demand for the specimen would be 22mm. At this displacement the maximum effective v-m strain in the plate was found to be 6.1%, where the curvature radius changes, as shown in Figure 6.32.

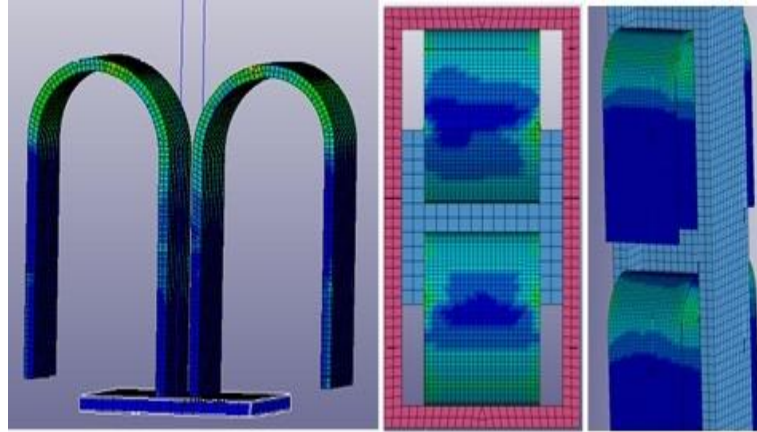


Figure 6.32 Strain distribution in one UFP set in the brace

It was experimentally concluded (Baird et al., 2014) that for strain values of 6.3%, no failure data were reported hence the researchers expected that the UFP could undergo 150 cycles before failure. In Figure 6.33 is presented the change in the post-yield stiffness of the system when different number of tendons are used, as previous research has found that one of the most important factors affecting residual drift is the post-yield stiffness of the inelastic mechanism (MacRae & Kawashima, 1997). As expected, the post-yield stiffness of the system is directly related to the prestress system, namely a higher tendon area or stiffness, causes an increase in the post-yield stiffness of the brace.

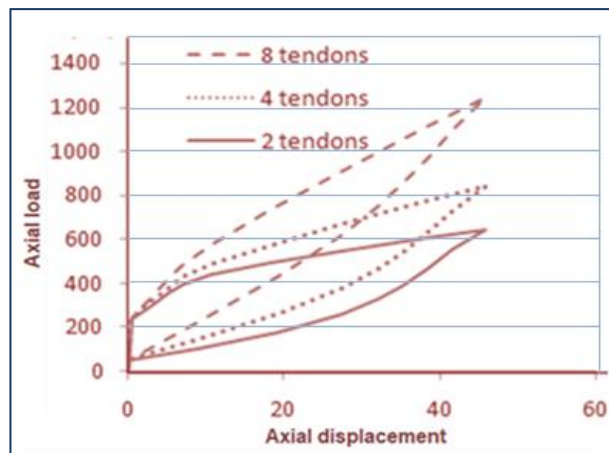


Figure 6.33 Load (kN)-displacement (mm) curve of Specimen 2 for different prestress systems

Load distribution in this brace and the moving yielding region within the UFP, makes the system more redundant than the previous friction based or yield based SCED braces, where concentrated forces arose in the friction mechanisms or buckling occurred in the yielding regions. A previous study (Xhahysa et al., 2017) compared a possible UFP SCED brace with a previously tested brace which consisted of a 7.86 m long dual core specimen, that had BRB (Buckling Restrained Brace) incorporated inside (Chou et al., 2016a, 2016b). In order to obtain the required load capacity, instead of the BRB in the original configuration, a set of 15 double UFPs was assumed. The set of UFPs could provide the necessary ultimate resistance and the displacement capacity, if they are to substitute the original BRB energy dissipating mechanism in the brace. On the other hand, a significant decrease in weight was obtained.

6.12 Conclusions

In this chapter the performance of an alternative brace was evaluated. At first the energy dissipation of the new implemented mechanism was analyzed, and next prestressing of the system was applied. This enhancement allows the brace to be installed in new structures as well in order to maximally reduce the residual deformation remaining in structures after being subjected to seismic loads.

- The new systems are more redundant than previous yield based SCED models, as the moving yielding region is distributed uniformly over many elements.
- Additive performance of UFP set shows that:
 - The yield displacement is the same for all specimens, not depending in the number of UFPs that are incorporated in the set analyzed.
 - Initial stiffness, yield and ultimate load are linearly proportional to the number of UFPs present in the set.
- Both braces with UFP energy dissipation mechanism analytically tested showed a flag shape hysteresis, confirming that the self-centering system is working properly.

- The brace maximum load when subjected to the same displacement in tension and compression is approximately the same, whereas previous models with core plates yielding in tension/compression, showed asymmetry during tension and compression cycles, due to multi-wave buckling of core plates causing friction between the core plates and the tubes.
- When compared to friction-based braces, the new system provides a more gradual stiffness change. This can possibly avoid the peak floor acceleration that was observed in frames that incorporated friction-based SCED braces.



CHAPTER SEVEN

RETROFIT SOLUTION: BRACE INTEGRATION IN THE FRAME

7.1 Introduction

In previous chapters, two main issues were addressed, namely the seismic performance level of the original RC precast frame structure, and generation of a new brace that can be an alternative retrofit strategy for this frame. Performance of the frame was characterized by low strength capacity and non-uniform stiffness distribution in height that generated the soft story mechanism. The next step is to choose a brace configuration for the retrofit scheme, such that both of these problems are solved.

7.2 Brace Pattern in the Frame

As preliminary design criteria for the braces, it is considered that these braces will carry 70% of seismic shear for each story and the arrangement of the braces in height shall generate an appropriate stiffness distribution to avoid the soft story mechanism. According to 4.3.3.2.2 (1)P EN 1998-1, 2004, the seismic base shear force F_b , for each horizontal direction in which the building is analyzed, shall be determined using the following expression:

$$F_b = S_d(T_1)m\lambda \quad (7.1)$$

$S_d(T_1)$ is the ordinate of the design spectrum at period T_1 equal to: $S_d(T_1) = 0.29$ for soil type D (for RP 72 years) or $S_d(T_1) = 0.44$ for soil type D (for RP 475 years). T_1 is the fundamental period of vibration of the building for the lateral motion in the direction considered, and is equal to $T_1 = 0.65s$. m - is the total mass of the building, according to 3.2.4 (2)P EN 1998 -1:2004 $m = 1185 \text{ kN}$. λ is the correction factor, the value of which is equal to $\lambda = 0.85$ as $T_1 < 2T_c$ and the building has more than two stories.

Base shear force for the design earthquake is:

$$F_b = S_d(T_1)m\lambda = 443\text{kN} \quad (7.2)$$

The distribution of the horizontal seismic force is done according to clause 4.3.3.2.3 EN 1998, with the formula as represented by Equation 7.3. The resulting shear forces for each story are presented in Table 1.

(7.3)

$$F_i = F_b \frac{z_i m_i}{\sum z_j m_j}$$

Table 7.1 Shear forces for each story

Story	Z_i (m)	m_i (kN)	$z_i m_i$ (kNm)	$z_i m_i / \sum z_j m_j$	F_i (kN)	V_i (kN)	$0.7V_i$ (kN)
5	14	237	3318	0.333	147.7	147.7	103.4
4	11.2	237	2654.4	0.267	118.1	265.8	186.1
3	8.4	237	1990.8	0.2	88.6	354.4	248.1
2	5.6	237	1327.2	0.133	59.1	413.5	289.4
1	2.8	237	663.6	0.067	29.5	443.0	310.1
S		1185	9954	1.0	443.0		

Design shear forces for each floor are determined. In this case the braces will be designed to carry 70% of the story seismic shear as a basic requirement for their preliminary design. It is recalled here that the horizontal force component of the brace should be equal to this value. The angle of the brace to the beam when placed in the second span of the frame is 27° . The theoretical respective forces required in each level brace are presented in Table 7.2.

Table 7.2 Brace properties for each story

Level	$0.7V_i$ (kN)	$F_{u, \text{brace}}$ (kN)	$F_{y, \text{brace}}$ (kN)	$K_{i, \text{brace}}$ (kN/mm)
5	103.4	116.0	76	15
4	186.1	208.8	138	27
3	248.1	278.4	184	37
2	289.4	324.8	214	43
1	310.1	348.0	231	46

However, as seen in the pushover analysis results, there is a soft story mechanism in the third story. The first two stories are very stiff, due to their higher reinforcement ratios when compared to the upper stories, hence despite the strength requirements of the brace, their respective stiffness is important as well. The braces cannot be distributed in a pattern that causes a uniform increase in stiffness from the last story to the ground story. Hence the frame is considered as composed of two sub-units.

- The first unit is composed of the upper three stories. Here the braces are arranged with uniform increasing stiffness and load capacity.
- Whereas in the first two stories lower stiffness is required in order to compensate for the original difference in stiffness between these units.

Hence the braces configuration for each story is chosen as shown in Figure 7.1.

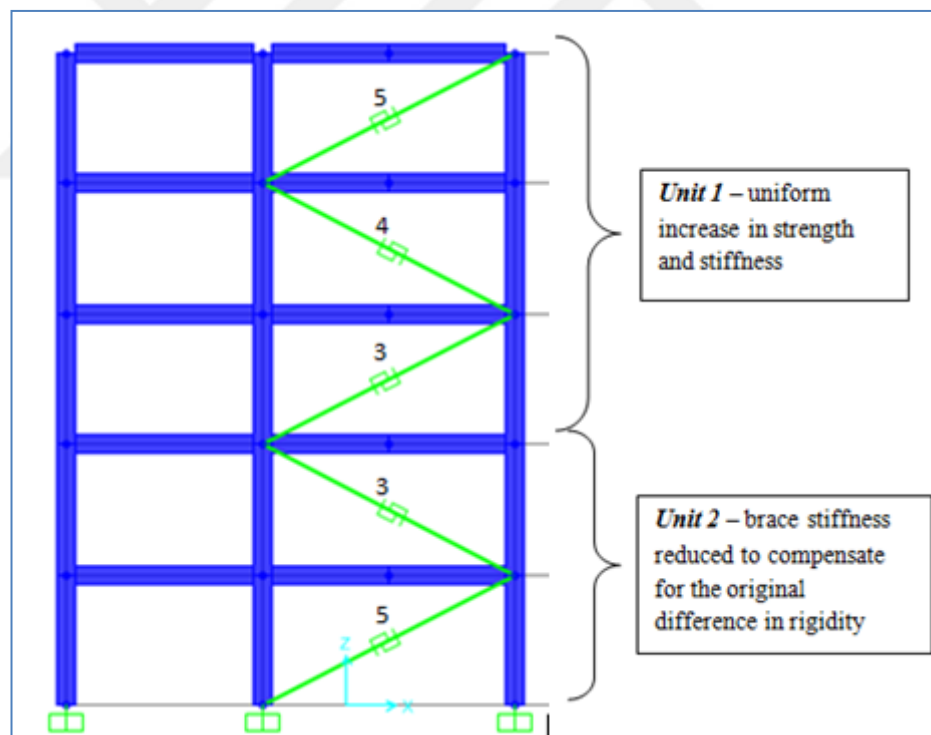


Figure 7.1 Brace configuration for each story

7.3 Global Acceptance Criteria

In the previous section, modal load pattern resulted in a lower frame capacity. Hence for this retrofit scheme it was chosen to use this load pattern and check the respective seismic performance enhancement in terms of strength, stiffness and ductility. As seen from the comparison of the two pushover curves for the strengthened and the original frame Figure 7.2, it is concluded that implementation of braces has significantly increased the strength ~ 3 times and no lateral strength reduction is observed up to 2.5% roof drift. The performance of the building is evaluated for the design earthquake. The strength and deformation limits recommended for this level of performance are selected based on the target objective as previously presented in Table 5.8.

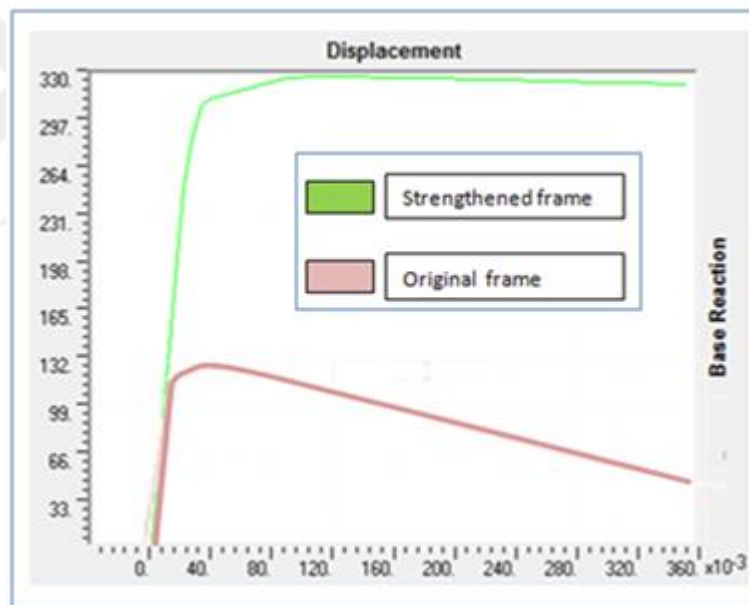


Figure 7.2 Base reaction (kN) - displacement (m) for original and strengthened frame

Once the demand spectra is defined, it is observed that in this case of demand and capacity the performance point occurs at the coordinates $(V,D) = (327, 0.107)$, as shown in Figure 7.3.

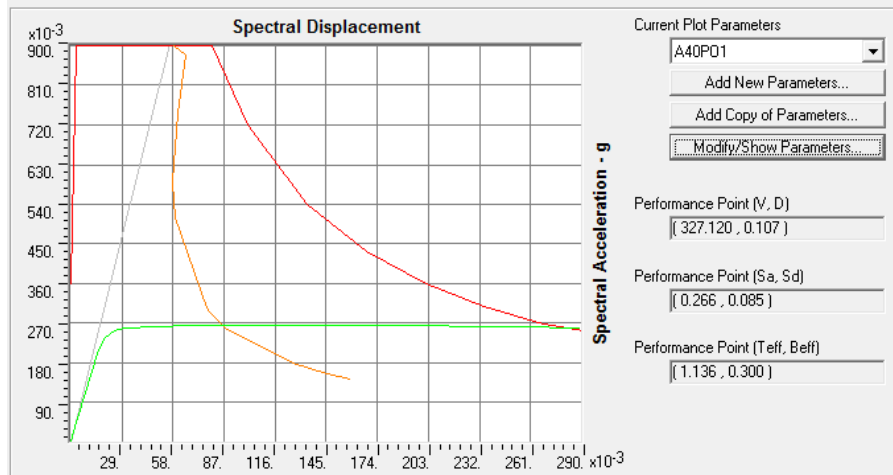


Figure 7.3 Performance point evaluation

At this point (step 9) no lateral strength degradation is observed. Each story drift values are presented in Table 7.3 for this performance point. Lateral deformations at the performance point displacement are checked against the deformation limits provided by ATC, 1996. Maximum total drift – the inter-story drift at the performance point displacement, is $d(\max)=0.01$ for floor 2, conform to the limit suggested. Whereas in the original frame drifts were in the range 0.021-0.023 for the 3 upper stories, not conforming to the limit suggested for the life safety level.

Table 7.3 Displacement and inter-story drift values

Floor	D (m) @ Step 9	Drift
Floor 5	0.1089	0.00253
Floor 4	0.1018	0.00742
Floor 3	0.081	0.00867
Floor 2	0.0567	0.01060
Floor 1	0.027	0.00964

7.4 Braces Activation

Braces incorporation in a structure, as a retrofit solution, despite providing the necessary strength and stiffness, should provide ample cycles of hysteresis, equivalent to abundant energy dissipation. Hence, braces have a crucial role when

they pass in the post-elastic phase. Their energy dissipation can protect the original structure members from failure. Therefore, another fundamental check for the retrofit option is the activation of braces.

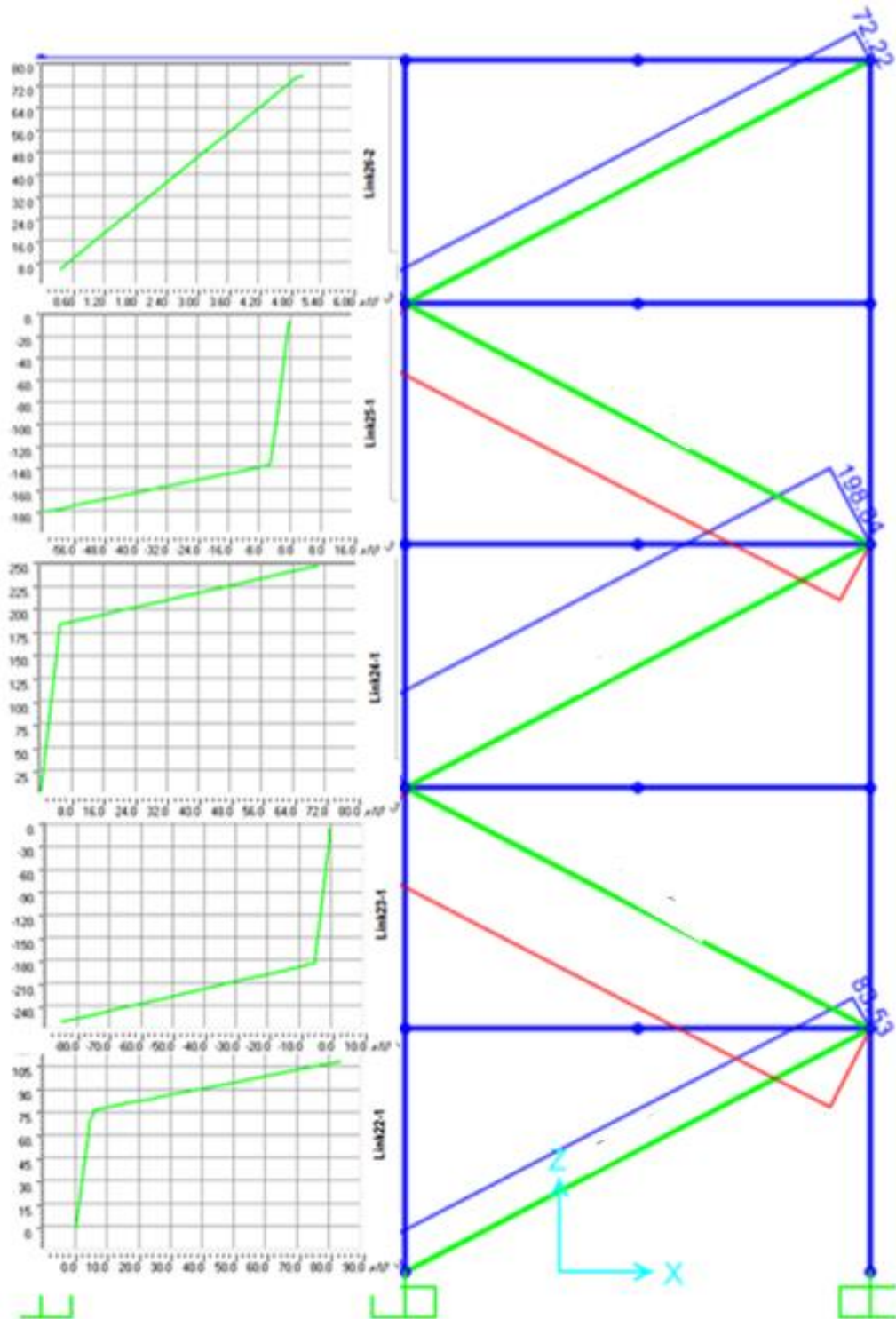


Figure 7.4 Axial forces in the braces in the performance point

At the performance point it is obvious that all braces are activated. In Figure 7.4 are presented the axial forces in the braces in the performance point, either in tension or compression, whereas on their left side are presented their respective hysteresis.

7.5 Hinge Pattern After Strengthening

Another important parameter for the performance evaluation of the frame is the hinge distribution pattern in the performance point. The optimal behavior for the hinges is to be in a life safety level in this stage, either in even a lower deformation level. In Figure 7.5 the hinges distribution with their respective performance levels is presented. It is clear that most of the hinges have just reached the yielding point, or are still in the linear phase. Only 28% of them are in the life safety level. Moreover, it can be highlighted here that most of these hinges that have undergone the largest deformation in the frame, are located at the beams. The rest is located mainly in the first story columns, as expected. This is a favorable plastic mechanism developed in the structural elements. The retrofit strategy changed the hinge distribution pattern. In the original frame hinges that had gone the largest deformation formed in the third story column, creating a soft-story mechanism, after retrofit this was solved.

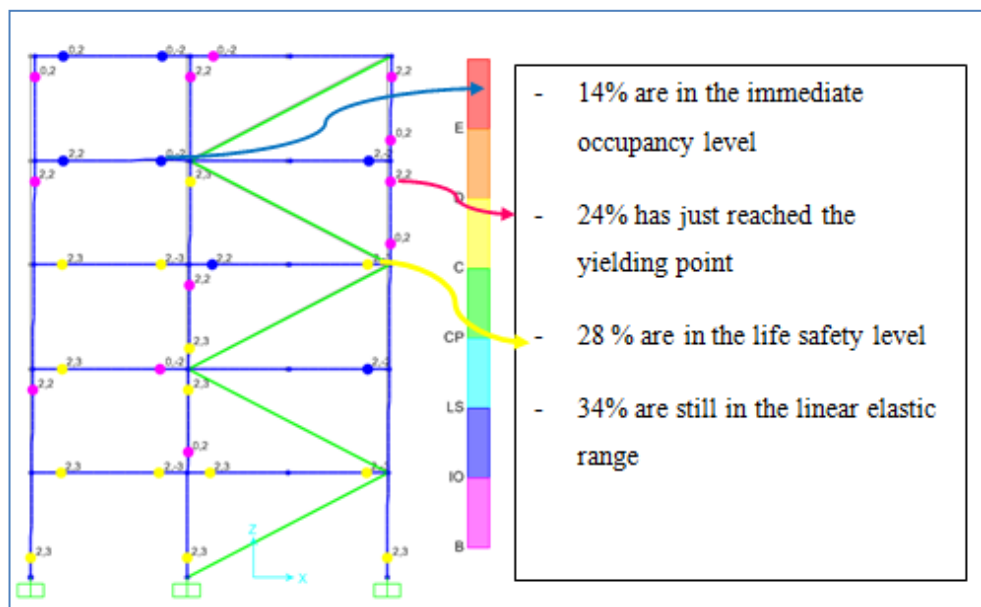


Figure 7.5 Hinge pattern in the performance point

Hence it can be concluded that in global and in local level as well the frame is within the life safety level of performance for the design earthquake, which was the initial objective.

7.6 Foundation Strengthening

The key reason behind the alternative brace selection as a retrofit system for the frame of interest was to avoid large additional stresses in the base. Implementation of very stiff strengthening elements in the retrofit scenario could result in large overturning forces in the existing foundation, that would require its retrofit as well. Foundation strengthening is not only an expensive process, but also cumbersome and time consuming.

In this perspective, as seen also from the pushover curve results, the general stiffness of the frame has not increased, but shear strength was significantly enhanced and the soft story mechanism was avoided as well, providing for the frame a general life safety performance level for the design earthquake. Slight increase in the base reactions was expected for this scenario. The pad foundation design details, provide as well the maximum axial force, moment and shear force that they were designed for. From these data and the pushover analysis results are generated the following Table 7.4.

Table 7.4 Base reactions after strengthening and respective allowable level

	Allowable level	Level after strengthening	% difference
Axial force (kN)	1106	795	-28
Moment (kNm)	163	186	14
Shear Force (kN)	85.8	93	8.3

As it is seen from the results in Table 7.4, the difference in the base reactions from the allowable limits is slight, when compared to the fact that the overall frame lateral resistance was increased up to ~3 times. In case a lower level of lateral resistance is chosen from the owner, in order to be still in the safety level for the design earthquake, these differences may become negligible. However, only the presence of

the details of the rest of the longitudinal frames incorporated in the full building model can accurately define the total stresses arising in the base.

7.7 Simulated 3D Building

In the project that was obtained from the archive, very few technical details were provided for the longitudinal frame of the building. In order to check the effect of the chosen strengthening scheme in a building, based on the few data provided in the project, is simulated a 3D structure with 2 spans in the transverse direction, and 5 spans in the longitudinal direction. The two peripheral outer transverse frames are strengthened with the technique previously presented. In Figure 7.6 are shown the original building and the retrofitted one.

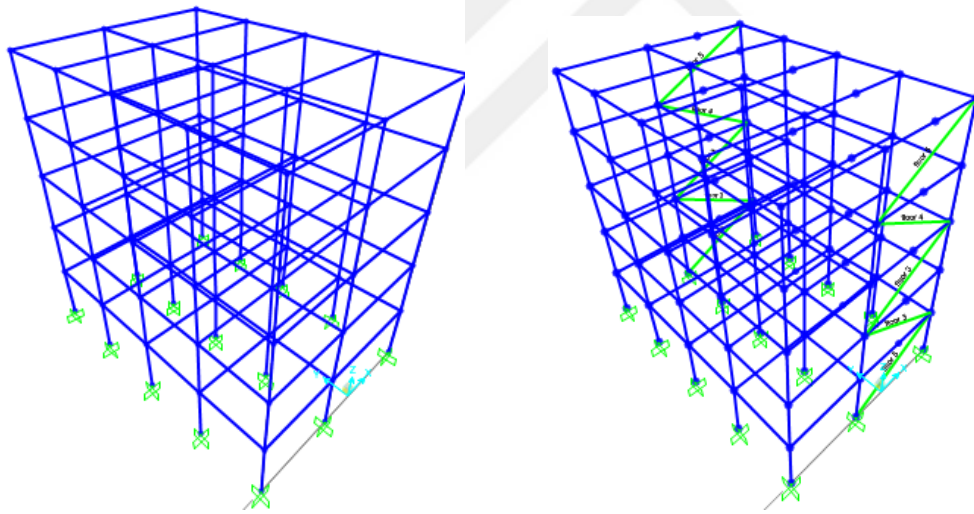


Figure 7.6 (a) Original structure (b) Retrofitted structure

The beam hinges are modeled in the frames of the transverse direction only, parallel to the loading direction. Hence, this analysis aims just to show a trend of the behavior in case a 3D building is retrofitted, not the exact performance level.

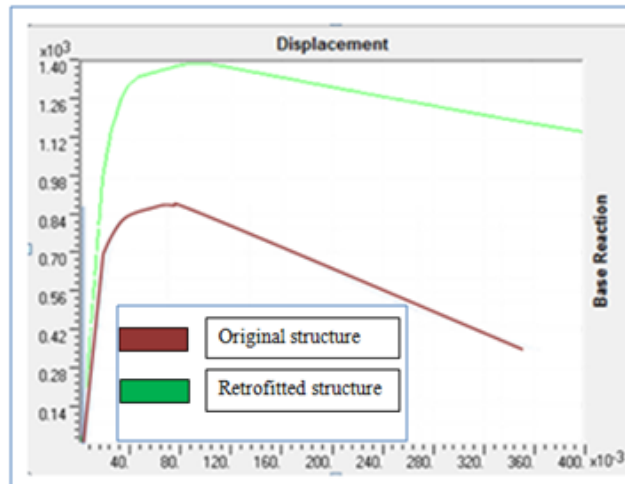


Figure 7.7 Capacity for original and retrofitted structure

From the superposition of the pushover curves, as apparent in Figure 7.7, it is clear that the strengthening scheme chosen increases considerably the lateral resistance, without interfering with stiffness, so that large overturning forces in the foundation are avoided.

7.8 Time History Analysis

7.8.1 Accelerogram Record Selection

In the previous sections it was analyzed the structure with static nonlinear procedures, evaluating the forces and stresses that arise in all elements in every pushover step. However, the most realistic type of analysis for structures subjected to seismic actions is the nonlinear dynamic analysis. Hence in this section are presented the data input and the respective results for this type of analysis. The two specimens analyzed, are the bare frame and the strengthened frame with the new developed braces.

In the dynamic analysis one of the most important parts is the selection of the accelerograms to be used as time-history curves. In absence of strong ground motion data records for the region we are interested, a very close region that belongs to Montenegro is chosen, namely Petrovac, as shown in Figure 7.8.



Figure 7.8. Maximum reference accelerations in Type A Soil (Aliaj et al., 2010)

The shape of the ground acceleration time history, chosen corresponds to the 24 s long strong phase of the N–S component of the ground acceleration record, recorded at Petrovac (Tomazevic, 2009) during the Montenegro earthquake of April 15, 1979, with maximum measured ground acceleration $a_{max} = 0.43g$. In fact, this is one of the strongest earthquakes that has hit Shkodra region, as shown in Table 7.5, hence it provides the most appropriate record to be used for this site.

Table 7.5 Earthquake properties and the evidenced phenomena (Aliaj et al., 2010)

Earthquake	M	I ₀	Location	Evidenced Phenomena
15/04/1979	6.9	IX	Montenegro Coast (District of Shkodra)	Intensive subsidence of ground, ground slides. Ground fissures with length 50-250m and width 40-50cm, accompanied by water and sand fountains, sand pseudovolcanoes, with diameter up to 1m, subsidence of river banks of Drini and Buna Rivers.

The response spectrum of the record is in general agreement with the EN 1998-1, 2004, proposed design spectrum with a flat part in the range of natural periods of vibration from 0.15 to 0.6 s, depending on the type of the ground. The period range

of the flat part of the design spectrum is in good agreement with the range of the first natural period of both frames, respectively: bare frame period $T=0.65s$ and strengthened frame period $T=0.5s$. In the structure time history analysis only the first 10s part of the accelerogram is input, as shown in Figure 7.9.

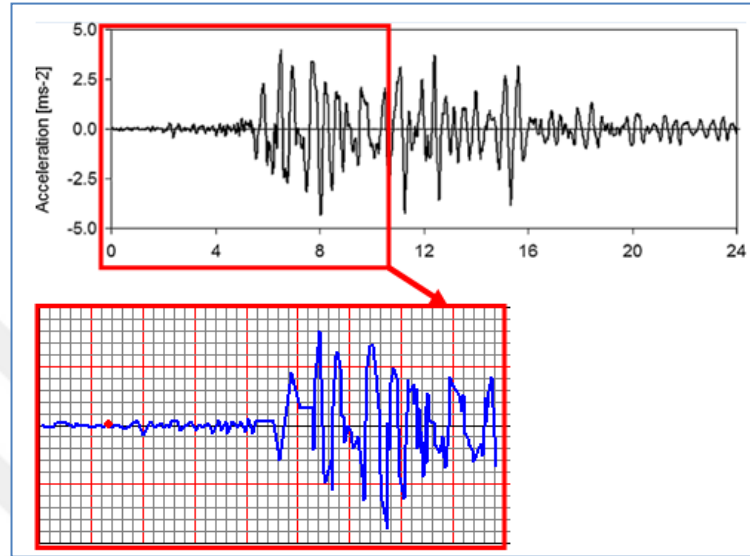


Figure 7.9 First 10s of the accelerogram record

7.8.2 Analysis Results

From the previous section analysis, it was obtained a general trend of the structure behavior, that consisted in strength increase up to 3 times and slight increase in stiffness. Moreover, the braces with the new configuration were able to avoid the soft story mechanism and provide a uniform drift distribution for the strengthened frame. Hence in this section some other local parameters are analyzed: braces hysteresis, roof displacement and acceleration. Moreover, axial forces, shear forces and moment in the most stressed column of the first story are presented.

a. Braces hysteresis

The main criteria for selection of the braces individual properties and general configuration were the low initial stiffness, but significant strength and ductility. In this perspective these braces were able to activate at low drifts, hence start dissipating energy quite early, by protecting the frame structural elements from

excessive stress. For the input ground motion, they provided ample cycles of energy dissipation in each story, as shown in Figure 7.10.

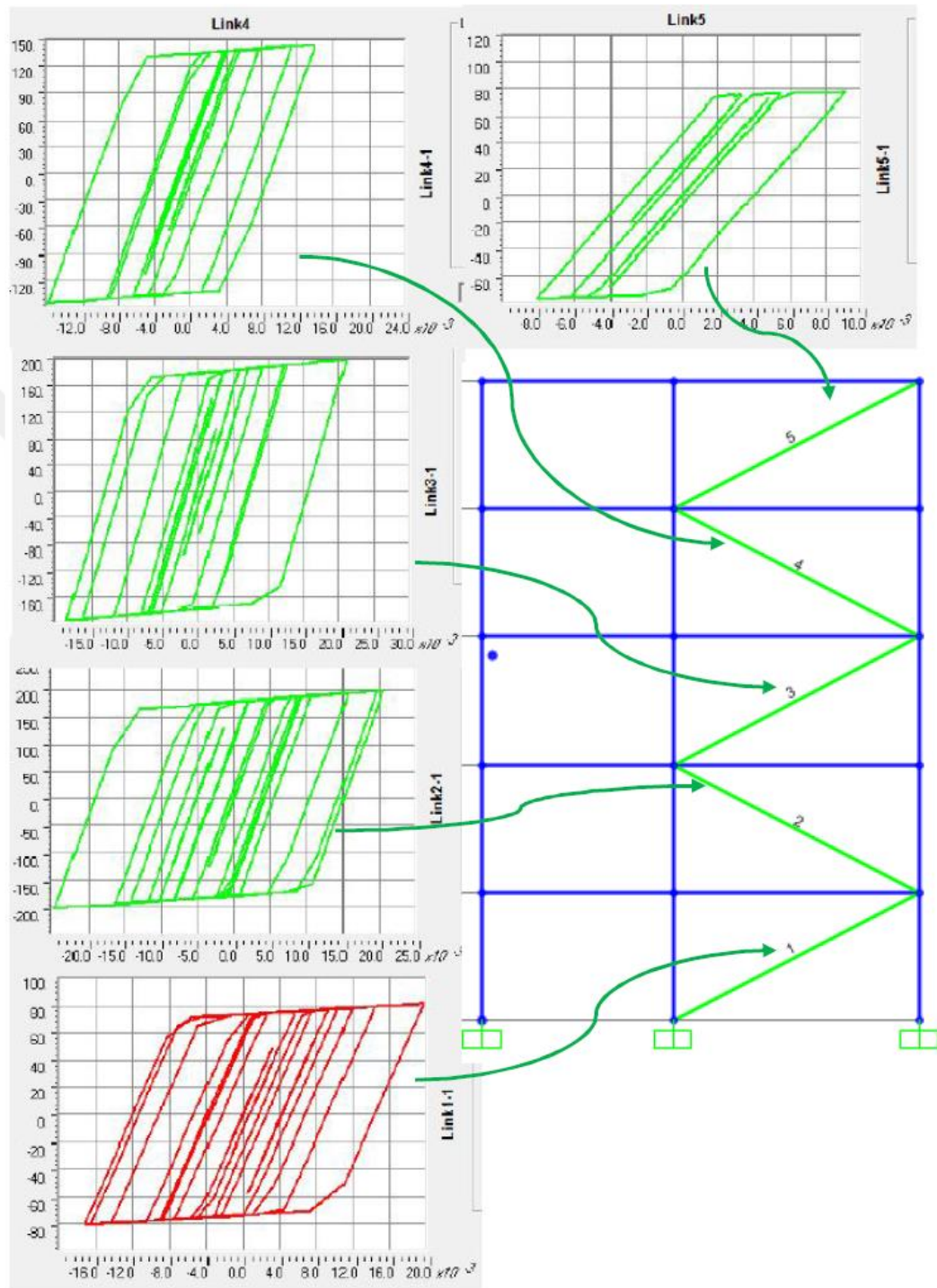


Figure 7.10 Brace hysteresis for every floor in the time history analysis

b. Roof displacement and acceleration

The energy dissipation provided by the braces, caused a significant reduction in the roof displacement and acceleration as shown in Figure 7.11 and Figure 7.12. In case a normal BRB brace would have been used, with the initial stiffness up to 3 times larger than the UFP brace, after strengthening smaller drifts would be expected, but there is chance that these braces implementation could increase the floor acceleration. In a previous study (Mayes & Goings, 2004), a new brace with lower stiffness than the current steel braces were designed by KaZaK Composites Inc. It consisted of an aluminum damping core element with lightweight composite filler material surrounded by a stiff outer shell to prevent buckling when loaded in compression. Similarly, to other buckling restrained braces it was designed to add stiffness and damping to a structure but the stiffness contribution was only 1/3 the stiffness (*same values to the new brace*) of steel dampers. The lower elastic modulus of aluminum compared to steel would increase the deformation before yield occurs for a structure of same strength. The study results (Mayes & Goings, 2004) showed as well in terms of the peak floor accelerations and floor response spectra, that the moment frame with the aluminum damper has the best performance, when compared to conventional steel BRB. However, the new developed brace in the currently mentioned study, when used for strengthening of the original frame, produced larger drifts in the structure. However, in the present case, the brace showed a significant decrease in roof drift and acceleration as well.

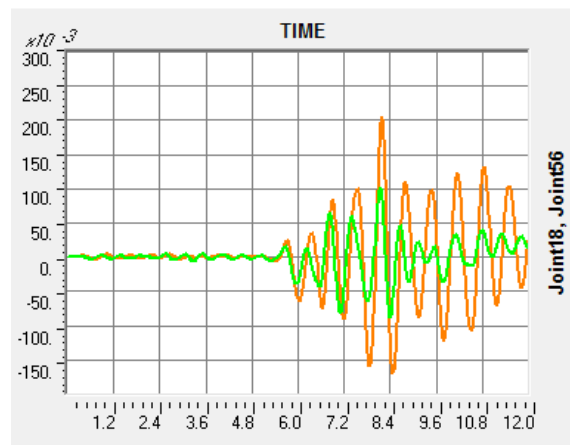


Figure 7.11 Roof displacement: (green) the strengthened frame; (orange) the original bare frame

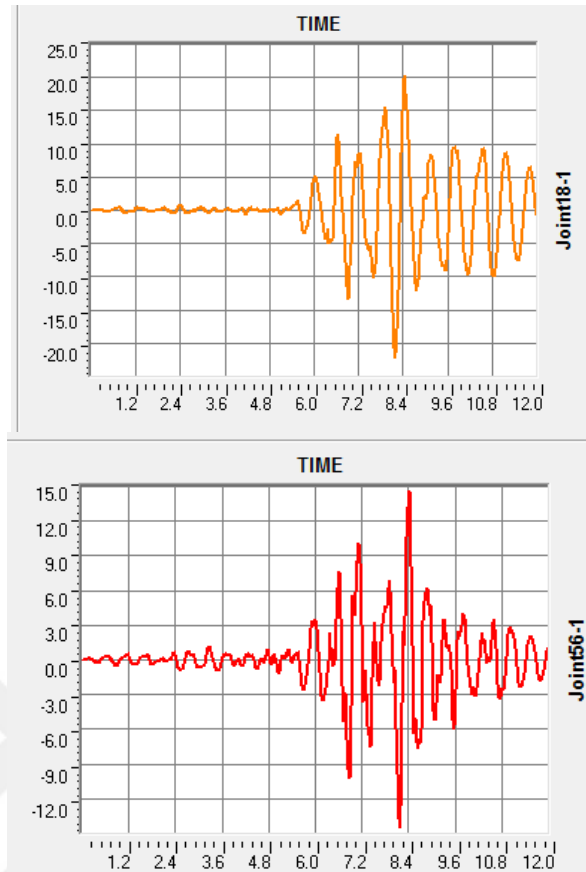
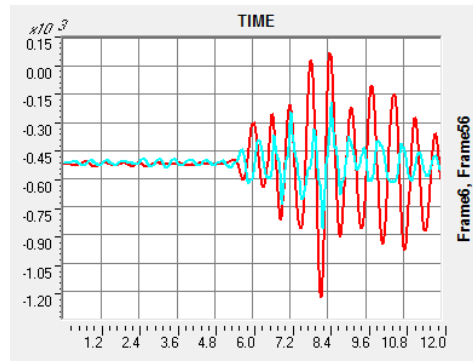


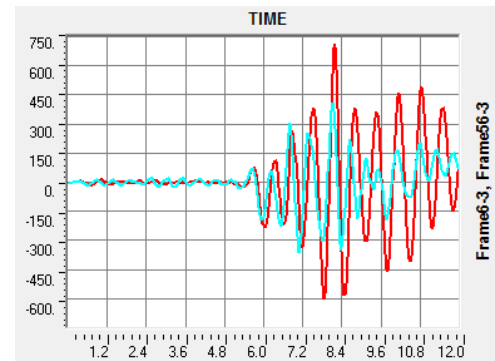
Figure 7.12 Roof acceleration (orange) bare frame and (red) strengthened frame

c. Axial force, shear force and moment in the most heavily loaded column

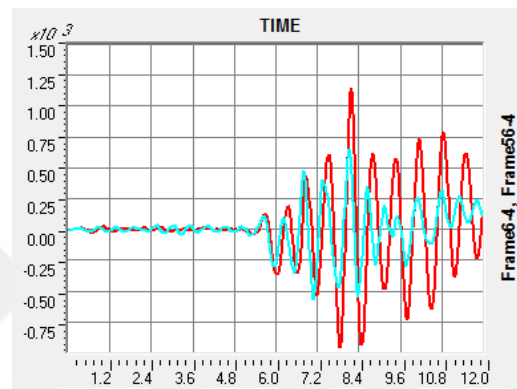
Braces, despite being a very good energy dissipating device, they are extra members that are stressed in order to act as fuses, and reduce the earthquake demand in the individual structural elements. Upon application of braces in the strengthened frame, the lateral resistance has increased up to 3 times, but the stresses coming from axial force, shear force and moment in the ground-story most heavily loaded column are decreased as shown in Figure 7.13. These enhanced results show again why the strengthening option chosen is the most optimal.



(a) Axial force



(b) Shear force



(b) Moment

Figure 7.13 Forces in the most loaded column for bare (red line) and strengthened frame (blue line)

CHAPTER EIGHT

CONCLUSIONS AND DISCUSSIONS

This study aimed to evaluate the seismic performance of a typical of RC precast structure that was designed for the highest seismic region in Albania. The high seismic activity of the country, the deficiencies of old design codes and seismic maps and the considerable number of reinforced concrete precast structures that were designed prior to the aseismic regulations came to force, highlight the importance of this study. An optimal seismic retrofit alternative is proposed in this work and the enhanced version of the new developed brace is analyzed for application in new structures as well. The major outcomes of this study are categorized in three major areas, respectively:

- Seismic performance evaluation
- Retrofit strategy and new brace development
- Structural performance after strengthening

In brief, these outcomes are summarized as follows:

- *Seismic performance evaluation*
 - The structures built prior to the current design code in Albania, despite having deficiencies in seismic detailing, were designed based in an unconservative seismic demand as well. It is judged as a fundamental need to update this design code with the probabilistic approach of the seismic parameter's definition, and consequently performing the necessary changes in the design code itself.
 - A new attenuation equation was used in this study to generate the peak ground acceleration values for different return periods. It was noticed that the average value of the Peak Ground Acceleration for the 10% in 50 years (0.346) is approximately same as in the reference material (0.32) (Aliaj et al., 2010), yet it is more conservative.

- Referring to the statistical data on the building stock (CENSUS, 2011), in total there are 187,633 residential buildings that were constructed before the last design code was applied. This is a significant number of in-habitated structures that need an immediate seismic performance evaluation.
- The RC precast frame was analyzed for possible seismic detail deficiencies. It is observed that the main beam reinforcement is continuous up to the column outermost face, and the lap splices in columns do not occur in the most stressed regions. These two characteristics were mainly observed as deficiencies in other countries for the old construction type, and were some of the main causes for the structure failure. However, the problem observed in these frames, is the confinement of elements. Neither the transverse reinforcement in the column, nor within the joint does not meet the recent criteria for seismic design. The socket foundation design is generally conforming to the criteria of the recent studies.
- The possible failure mode of the columns was checked. No brittle shear-type failure is expected. It resulted that most of the columns show a ratio more than 1. Only 3 columns have a ratio slightly less than 1(0.96). This occurred because when considering the cracking shear strength of the cross section, in order to be conservative, it is neglected the positive effect of the axial compression. Hence all the columns are expected to show a flexural type of failure, and this is how they are modeled for the pushover analysis.
- The beam joint is expected to allocate the yielding beyond the connection region; hence it can be considered as a strong connection. In these conditions yielding is expected in the region between the column face and the precast joint, hence no concentrated damage is expected.
- Nonlinear hinge properties generation was based on an analytical approach. However, the results were also compared to the suggested values in FEMA-

356 and the values were either slightly changing as in the case of beams, or fully compatible as in the case of column.

- Two load patterns were used for the pushover analysis of the frame, uniform and modal. Despite the discrepancy in the base shear values, as observed also in previous research, in both cases two important common properties were observed, such as:
 - The frame has lost more than 20% of their lateral resistance before the performance point is reached.
 - At the last cycles formation of soft story at the same level starts.
- Pushover results for both loading patterns of the frame show that for the provided demand, there is not enough strength and deformation capacity in the frame to obtain at least a life safety performance level as required. In these circumstances retrofit of the structure is needed.
 - *Retrofit strategy and new brace development*
- The main criteria that were considered before choosing the retrofit strategy are summarized as follows:
 - The structures are of residential function, and the occupation cannot be disrupted so a fast application technique should be chosen. Moreover, as the traffic patterns have been already set in the structure, we cannot cause full closure of any frame in case shear wall is chosen.
 - Stiffness enhancing is not required through all the height of structure, but should target the last three floors where the soft story mechanism is observed. As the best alternative should be to strengthen the structure, without causing any large stresses in the foundation that may require their strengthening as well.

- For all the requirements that aim the optimal strengthening scenario, it is concluded that “braces” provide the best option. However, the strengthening system that shall involve braces should provide for the structure enough energy dissipation as well.
- Common braces with large capacity should be applied to systems that have a significant lateral deformation capacity, in order to become active. In this case the RC frame is not expected to show significant ductile behavior, hence braces that are early- activated are needed.
- Dissipaters using direct shear, tension, compression mechanisms are less suitable, as they tend to generate large strain demands. Whereas flexural yielding, which is the leading working mechanism in the UFP plate, uses more indirect load paths, lowering the strain demand, hence resulting in larger displacement capacity. Therefore, a new brace composed of a series of UFP plates, placed longitudinally between two concentric tubes that move relative to each other by dissipating energy is suggested in this study.
- Buckling Restrained Braces with identical specified strengths may have stiffnesses that vary by as much as 20% due to differences in the overall length or the length of internal sections of the brace, which may result in unintentional system behavior. Whereas the new developed UFP braces have an initial stiffness dependent just in UFP number and dimensions, and not the brace length.
- From experimental results in a single UFP, there is an over-strength factor of approximately 185%. The suggested range for overstrength in UFP values by previous research is 145-215%. Whereas in BRB the strain hardening factor varies generally up to ~1.6. Both the braces show similar behavior in this aspect.

- When UFP are used for brace, the yielding occurs at 5mm displacement (for a typical UFP configuration), no matter what the number of UFPs is used in the brace. When is considered the normal brace length 5-7m, it means that this new brace yields at a range of tensile strain 0.07% to 0.1%, which means that these braces are activated quite early and can be applicable to buildings with low ductility.
- The UFP brace has slightly better performance in terms of symmetric behavior of the hysteresis when compared to BRB. This can make it a favorable option of energy dissipation in self-centering braces.
- The distribution of yielding regions in a member is the main parameter showing that this element will have redundant behavior when subjected to loading. A brace composed of UFP sets will have distributed yielding regions in all elements, and the yielding point within the UFP itself moves along the plate. This is a more favorable behavior, when compared to multi-wave buckling occurring in the core plates of common braces yielding in tension and compression.
- Materials with different annealing temperature were analyzed for a UFP set behavior and the results suggest that:
 - With increasing annealing temperature, the yielding points and the ultimate force decreases, hence under the same force level, the specimens undergo larger ultimate displacements;
 - With increasing annealing temperature under the same force level, the energy dissipation increases;
 - The initial and the post elastic stiffness of all specimens is approximately same.
- The brace behavior with many UFP sets incorporated is additive:
 - The ultimate load of the brace is proportional to the number of UFP sets in a brace.

- The initial stiffness of a brace composed of a set of UFPs is linearly proportional to the initial stiffness of the single UFP set.
- Yield displacement is the same for all specimens;
- For self-centering braces, when compared to the friction-based system, the UFP alternative for an energy dissipation system, provides a more stable and reliable behavior, it does not require slotting the outer member, avoids the eccentricity of the friction interface bolts with respect to tube surface and the large concentration of forces.
- The alternative energy dissipation mechanism composed of a set of UFPs, can provide the necessary ultimate resistance and the displacement capacity to substitute the Buckling Restrained Brace in the original self-centering brace. Moreover, it provides additional benefits, such as redundancy, which make this alternative, a cost-effective solution.
- When UFPs are used as energy dissipation device in self centering braces:
 - The prestressed system of tendons for the UFP brace system has satisfactorily reached the flag- shape hysteresis.
 - The prestress force chosen for the current case is enough to bring the system back to zero deformation after the ultimate load of the energy dissipation system has been reached. The tendons are always in tension.

- ***Structural performance after strengthening***

- The implementation of braces has significantly increased the strength ~3 times and no lateral strength reduction is observed up to 2.5% roof drift.
- Maximum total drift is conforming to the limits suggested by ATC, 1996. Whereas in the original frame drifts were in the range 0.021-0.023 for the 3 upper stories, not conforming to the limit suggested for the life safety level.

- Braces have a crucial role when they pass in the post-elastic phase. Their energy dissipation can protect the original structure members from failure. Therefore, another fundamental check for the retrofit option was the activation of braces. At the performance point it is obvious that all braces are activated.
- After strengthening, at the performance point most of the hinges have just reached the yielding point, or are still in the linear phase. Moreover, most of these hinges that have undergone the largest deformation in the frame, are located at the beams. The rest is located mainly in the first story columns, as expected. This is a favorable plastic mechanism developed in the structural elements. In the original frame the most deformed hinges were located in the third story column, creating a soft-story mechanism. After retrofit this scenario changed, because the retrofit strategy improved the hinge distribution pattern. The strengthened frame is within the life safety level of performance for the design earthquake.
- After strengthening, the difference in the base reactions from the allowable limits is slight, when compared to the fact that the overall frame lateral resistance was increased up to ~3 times.
- A 3D structure with 2 spans in the transverse direction, and 5 spans in the longitudinal direction, with the two peripheral outer transverse frames strengthened, showed that the strengthening scheme chosen increases considerably the lateral resistance, without interfering with stiffness, so that large overturning forces in the foundation are avoided.
- The energy dissipation provided by the braces, caused a significant reduction in the roof displacement and acceleration.
- Upon application of braces in the strengthened frame, the lateral resistance has increased up to 3 times, but the stresses coming from axial force, shear

force and moment in the ground-story most heavily loaded column are decreased. These enhanced results show again why the strengthening option chosen is the most optimal.

FURTHER RESEARCH

Despite the many positive outcomes of this study, still there are some issues that have to be further analyzed and studied, such as:

- Frame joint experimental testing should be performed, so that after its behavior is experimentally evaluated, the best brace connection method to the frame can be chosen, in order to maximally reduce the stress concentration.
- Experimental testing of the new developed brace is needed in order to verify the results obtained analytically and evaluate its possible application in real scale buildings.
- More details about the longitudinal frame of the building should be provided, so that the other direction of the structure can be accurately analyzed.

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APPENDICES

APPENDIX A: CENSUS 2011 ALBANIA- STATISTICAL DATA

APPENDIX B: EARTHQUAKE CATALOGUE

APPENDIX C: COLUMN HINGE DATA

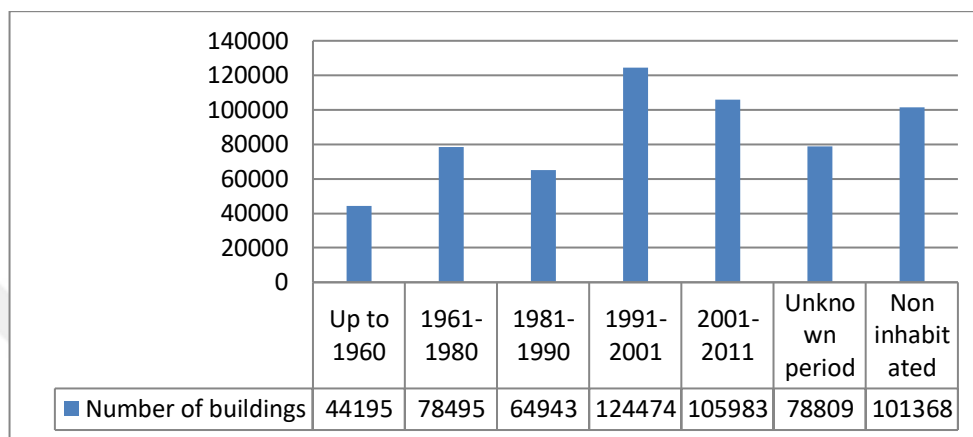
APPENDIX D: LS DYNA REDUCED KEYWORD AND MODEL DETAILS

APPENDIX E: PRECAST RC FRAME DETAILS

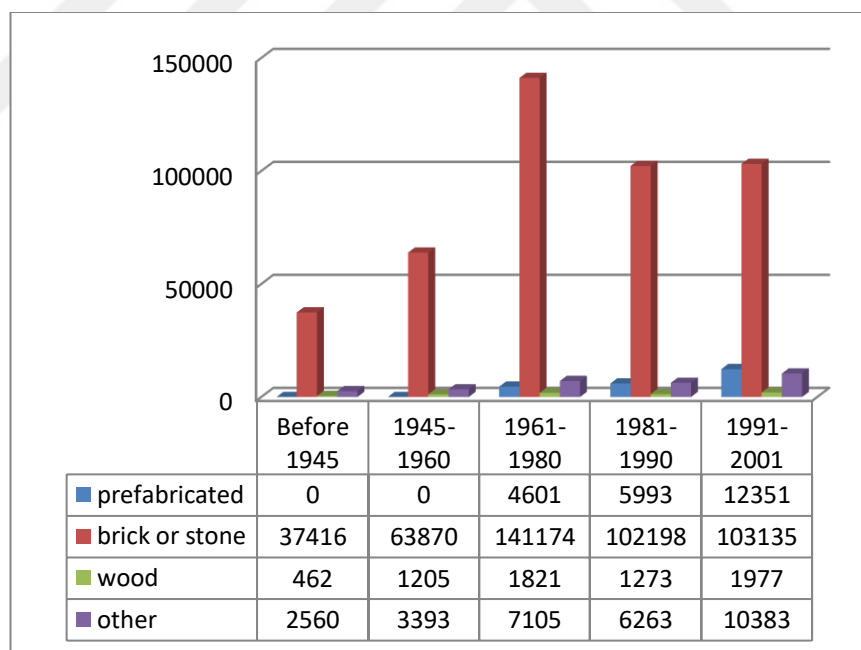
APPENDIX F: UFP BRACE DETAILS WITH/OUT SELF-CENTERING

APPENDIX A: CENSUS 2011 ALBANIA- STATISTICAL DATA

CENSUS 2011 – STATISTICAL DATABASE: Number of buildings constructed in a specific period and classification based on the building main construction material type.



(a)



(b)

Figure A.1 Statistical database

APPENDIX B: EARTHQUAKE CATALOGUE

Table B.1 Earthquake catalogue

Location (°)		Ms
42	19.5	6.6
42.2	19.8	5.5
42	19.5	6
42	19.6	5.4
42.1	19.6	5.9
42	19.5	5.5
42	19.5	5.3
42.3	21.3	5.7
42.1	20.5	5
42	19.5	5.5
42.1	19.5	5
41.9	20.8	5.8
42	21.4	6.1
42.2	18.8	5.3
42.2	18.9	5.2
42.3	18.9	5.2
42.2	19	5
42.2	18.9	5.8
42.1	18.8	5.3
42.1	18.7	5.2
42.2	19.9	5.5
42	19.3	5.5
42.3	19.2	5.5
42.3	19.1	5.3
41.92	20.14	5.1
41.94	19.06	5.2
42.01	19.1	5.4
42.16	18.87	6.9
41.87	19.2	5.2
42.29	18.6	6
41.94	19.2	5.4
42.31	18.75	5
42.24	18.7	6.4
41.84	19.49	5
42.14	19.13	5.1

In this table are registered all earthquakes that occurred in the period 1900-2005 (which are instrumentally recorded) located within the coordinates 41.87-42.31, 19.39-19.82 (Shkodra region).

APPENDIX C: COLUMN HINGE DATA

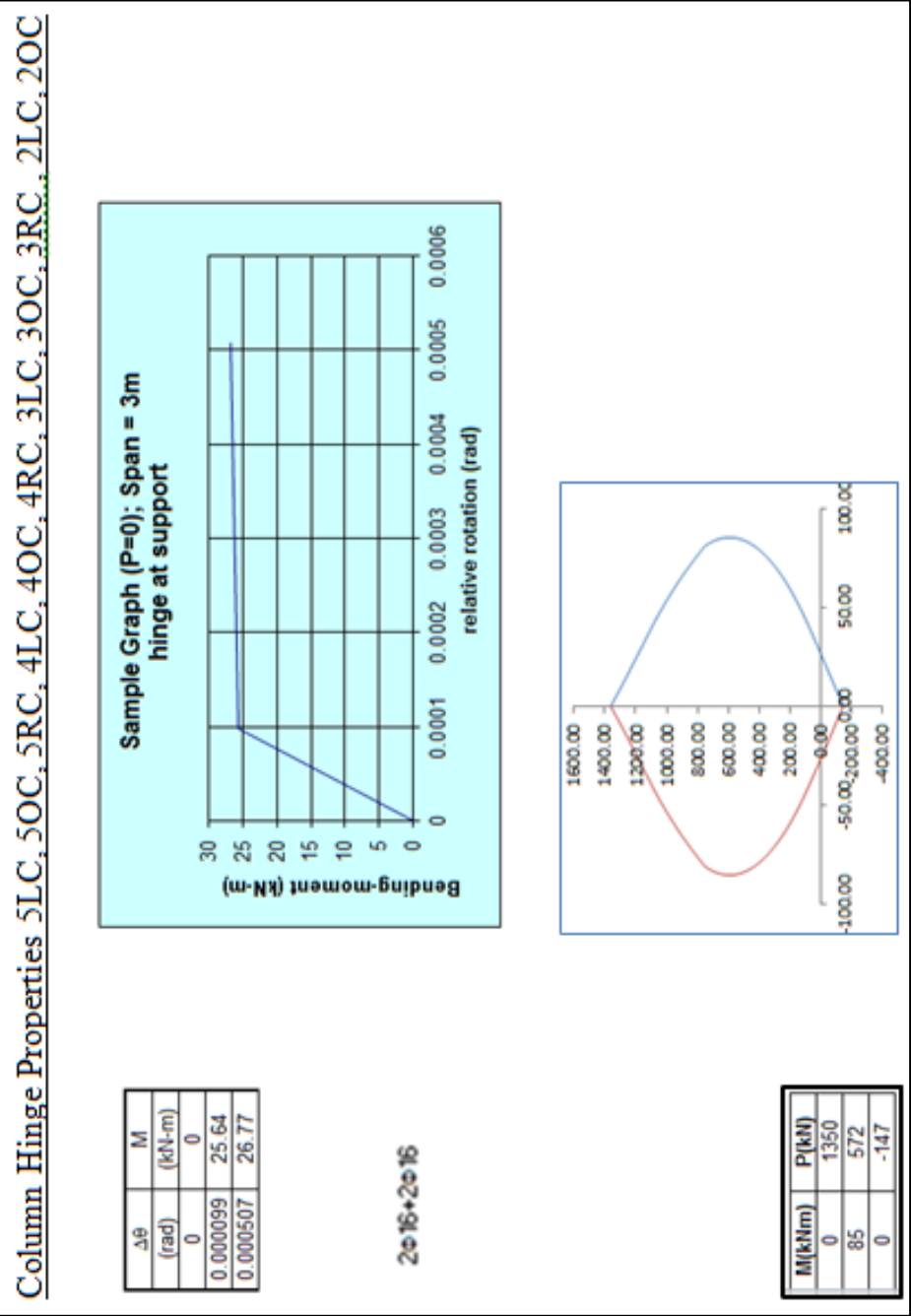


Figure C.1 Column hinge properties (a)

Column Hinge Properties 2RC

$\Delta\theta$ (rad)	M (kN-m)
0	0
0.000114	64.31
0.000545	67.22

4ϕ18+4ϕ13

M(kNm)	P(kN)
0	-371
126	546
0	1560

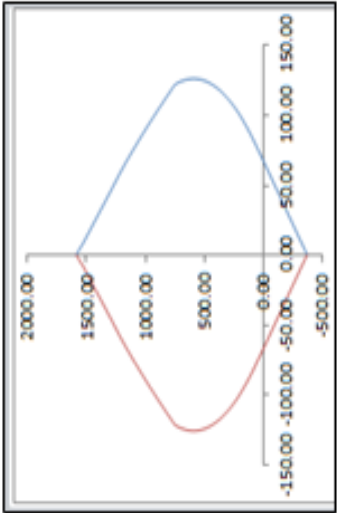
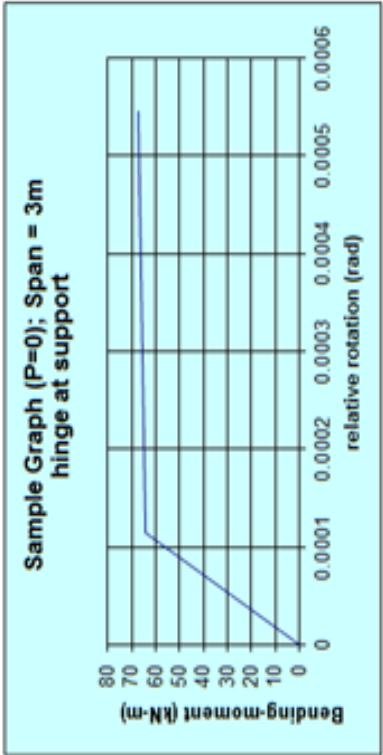


Figure C.2 Column hinge properties (b)

Column Hinges 1OR

$\Delta\theta$ (rad)	M (kN-m)
0	0
0.000120	121.43
0.000528	126.73

$2(4 \times 18 + 2 \times 24)$

M(kNm)	P(kN)
0	-702
185	572
0	1876

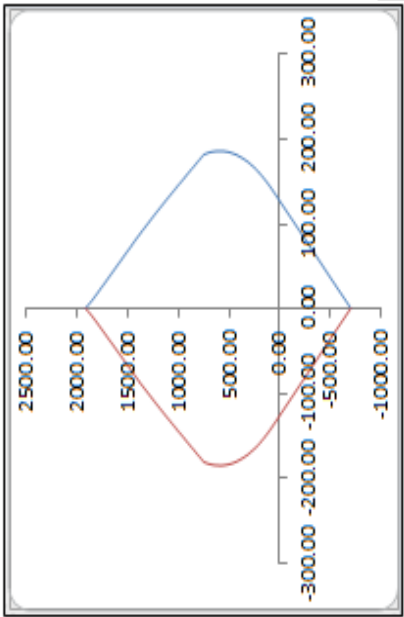
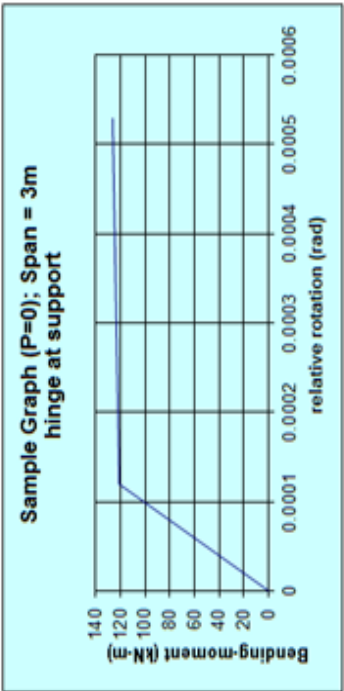


Figure C.3 Column hinge properties (c)

Column Hinges 1LC

$\Delta\theta$ (rad)	M (kN-m)
0	0
0.000109	129.03
0.000475	134.06

$$2(2 \times 18 + 2 \times 20 + 2 \times 24)$$

M(kNm)	P(kN)
0	-743
192	521
0	1894

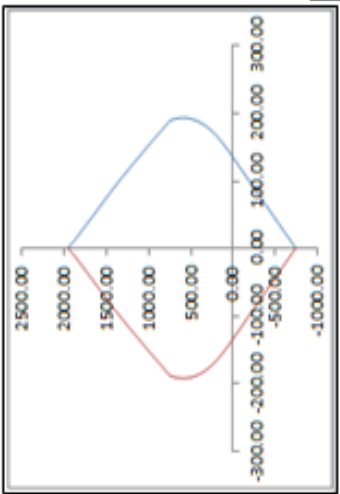
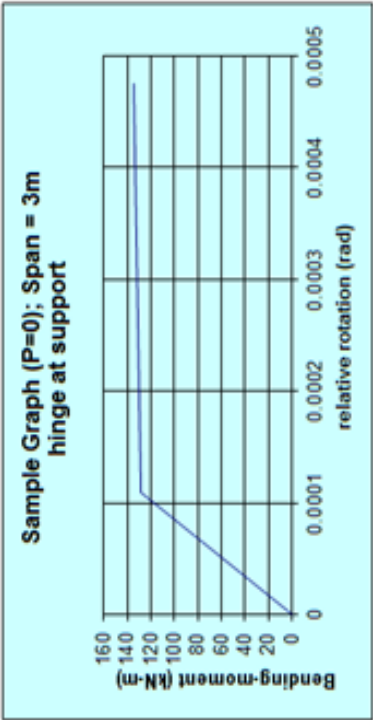


Figure C.4 Column hinge properties (d)

APPENDIX D: LS DYNA REDUCED KEYWORD AND MODEL DETAILS

In this section is presented the reduced LS DYNA keyword for the mini-brace subjected to displacement-controlled loading.

```

$# LS-DYNA Keyword file created by LS-PrePost(R) V4.3 -
30Oct2016(20:00)
$# Created on Oct-02-2017 (17:06:19)
*KEYWORD
*TITLE
$#
title
LS-DYNA keyword deck by LS-PrePost

*CONTROL_TERMINATION
$#   endtim   endcyc   dtmin   endeng   endmas
      2630.0         0       0.0       0.0  1.000000E8

*DATABASE_BNDOUT
$#   dt   binary   lcur   iopt
      1.0         1       0       1

*DATABASE_GLSTAT
$#   dt   binary   lcur   iopt
      1.0         1       0       1

*DATABASE_JNTFORC
$#   dt   binary   lcur   iopt
      1.0         1       0       1

*DATABASE_MATSUM
$#   dt   binary   lcur   iopt
      1.0         1       0       1

*DATABASE_NODFOR
$#   dt   binary   lcur   iopt
      1.0         1       0       1

*DATABASE_NODOUT
$#   dt   binary   lcur   iopt   option1   option2
      1.0         1       0       1       0.0       0

*DATABASE_RCFORC
$#   dt   binary   lcur   iopt
      1.0         1       0       1

*DATABASE_SECFORC
$#   dt   binary   lcur   iopt
      1.0         1       0       1

*DATABASE_SPCFORC
$#   dt   binary   lcur   iopt
      1.0         1       0       1

*DATABASE_BINARY_D3PLOT
$#   dt   lcdt   beam   npltc   psetid
      1.0         0       0       0       0
$#   iopt
      0

```

*BOUNDARY_PRESCRIBED_MOTION_RIGID

\$#	pid	dof	vad	lcid	sf	vid	death	birth
	27	2	2	1	1.0	0	1.00E28	0.0

*BOUNDARY_SPC_SET

\$#	nsid	cid	dofx	dofy	dofz	dofrx	dofry	dofrz
	43	0	1	1	1	1	1	1

*BOUNDARY_SPC_SET

\$#	nsid	cid	dofx	dofy	dofz	dofrx	dofry	dofrz
	42	0	1	0	1	1	1	1

*CONTACT_AUTOMATIC_SURFACE_TO_SURFACE_ID

\$#	cid								
	6								
\$#	ssid	msid	sstyp	mstyp	sboxid	mboxid	spr	mpr	
	27	7	3	3	0	0	0	0	
\$#	fs	fd	dc	vc	vdc	penchk	bt	dt	
	0.0	0.0	0.0	0.0	0.0	0	0.0	1.00E20	
\$#	sfs	sfm	sst	mst	sfst	sfmt	fsf	vsf	
	1.0	1.0	0.0	0.0	1.0	1.0	1.0	1.0	

*SECTION_SOLID

\$#	secid	elform	aet
	1	1	0

*MAT_KINEMATIC_HARDENING_TRANSVERSELY_ANISOTROPIC_TITLE

yoshida								
\$#	mid	ro	e	pr	r	hclid	opt	
		137E-6	200.0	0.3	0.0	0	0	
\$#		cb	y	sc	k	rsat	sb	h
		0.277	0.255	0.338	7.9	0.196	0.195	0.5
\$#		ea	coe	iopt	c1	c2		sc2
		0.0	0.0	0	0.0	0.0		0.0

*PART

\$#								
	Solid8							
\$#	pid	secid	mid	eosid	hgid	grav	adpopt	tmid
	8	1	13	0	0	0	0	0

*PART

\$#								
	Solid9							
\$#	pid	secid	mid	eosid	hgid	grav	adpopt	tmid
	9	1	13	0	0	0	0	0

*PART

\$#								
	Solid13							
\$#	pid	secid	mid	eosid	hgid	grav	adpopt	tmid
	13	1	3	0	0	0	0	0

```

*MAT_RIGID_TITLE
RIGID
$# mid ro e pr n couple m alias
    37.00E-6 200.0 0.3 0.0 0.0 0.0
$# cmo con1 con2
    0.0 0 0
$# lco or a1 a2 a3 v1 v2 v3
    0.0 0.0 0.0 0.0 0.0 0.0

*PART
$#
title
Solid25
$# pid secid mid eosid hgid grav adpopt tmid
    25 1 13 0 0 0 0 0

*PART
$#
title
Solid27
$# pid secid mid eosid hgid grav adpopt tmid
    27 1 3 0 0 0 0 0

*MAT_PLASTIC_KINEMATIC
$# mid ro e pr sigy etan beta
    17E-6 200.0 0.3 0.5 0.00689 0.0
$# src srp fs vp
    0.0 0.0 0.2 0.0

*MAT_PIECEWISE_LINEAR_PLASTICITY_TITLE
A36 ANNEALED 80 F
$# mid ro e pr sigy etan fail tdel
    47.83E-6 200.0 0.29 0.19987 0.0 0.0 0.0
$# c p lcsc lcsr vp lcf
    0.0 0.0 3 0 0.0 0
$# eps1 eps2 eps3 eps4 eps5 eps6 eps7 eps8
    0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0
$# es1 es2 es3 es4 es5 es6 es7 es8
    0.0 0.0 0.0 0.0 0.0 0.0 0.0 0.0

*DEFINE_CURVE_TITLE
A36 ANNEALED 80F
$# lcidsidr sfa sfo offa offo dattyp lcint
    3 0 1.0 1.0 0.0 0.0 0 0
$# a1 o1
    0.0 0.199874
    0.006246 0.289557
    0.014723 0.334728
    0.0203 0.354294
    0.02677 0.372321
    0.042385 0.404742
    0.06157 0.433448
    0.072501 0.446706
    0.09704 0.471448
    0.110648 0.483055
    0.14054 0.504983
    0.174002 0.52544
    0.211034 0.544656

```

0.230888	0.553856
0.262342	0.567197
0.307404	0.584213
0.356036	0.600441
0.408237	0.615971
0.464007	0.630875
0.523346	0.645214
0.586254	0.65904
0.652732	0.672399
0.722779	0.685329
0.796396	0.697865
0.873582	0.710036
0.954337	0.721868
1.038661	0.733385
1.082161	0.739032

*DEFINE_CURVE_TITLE

Disp

\$#	lcid	sidr	sfa	sfo	offa	offo	dattyp	lcint
	12	0	1.0	1.0	0.0	0.0	0	0
\$#		a1				o1		
		0.0				0.0		
		2.0				60.0		
		4.0				0.0		
		6.0				-60.0		
		8.0				0.0		

*SET_NODE_LIST_TITLE

apply load

\$#	sid	da1	da2	da3	da4	solver
	44	0.0	0.0	0.0	0.0	MECH

*CONSTRAINED_EXTRA_NODES_SET

\$#	pid	nsid	iflag
	13	12	0

*CONSTRAINED_EXTRA_NODES_SET

\$#	pid	nsid	iflag
	13	13	0

*CONSTRAINED_EXTRA_NODES_SET

\$#	pid	nsid	iflag
	13	14	0

*CONSTRAINED_EXTRA_NODES_SET

\$#	pid	nsid	iflag
	13	15	0

*CONSTRAINED_EXTRA_NODES_SET

\$#	pid	nsid	iflag
	27	19	0

*CONSTRAINED_EXTRA_NODES_SET

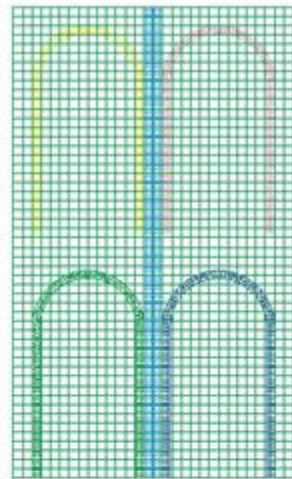
\$#	pid	nsid	iflag
	27	20	0

*CONSTRAINED_EXTRA_NODES_SET

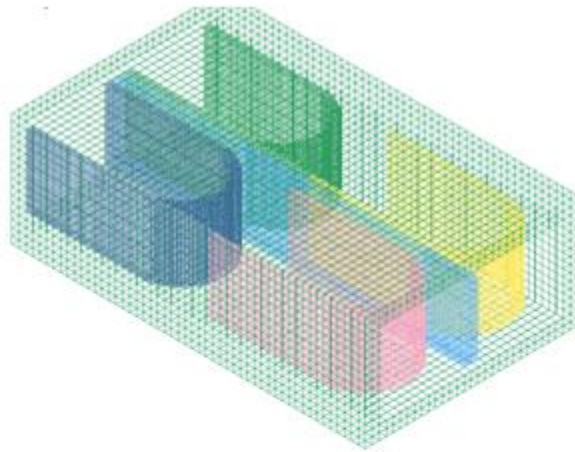
\$#	pid	nsid	iflag
	27	21	0

*CONSTRAINED_EXTRA_NODES_SET

\$#	pid	nsid	iflag
	27	22	0



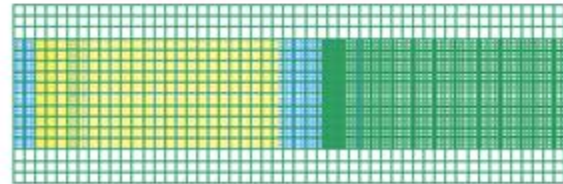
(a) Top view



(b) Isometric view



(c) Right view



(d) Front view

Figure D.1 UFP mini brace without self-centering – LS DYNA model

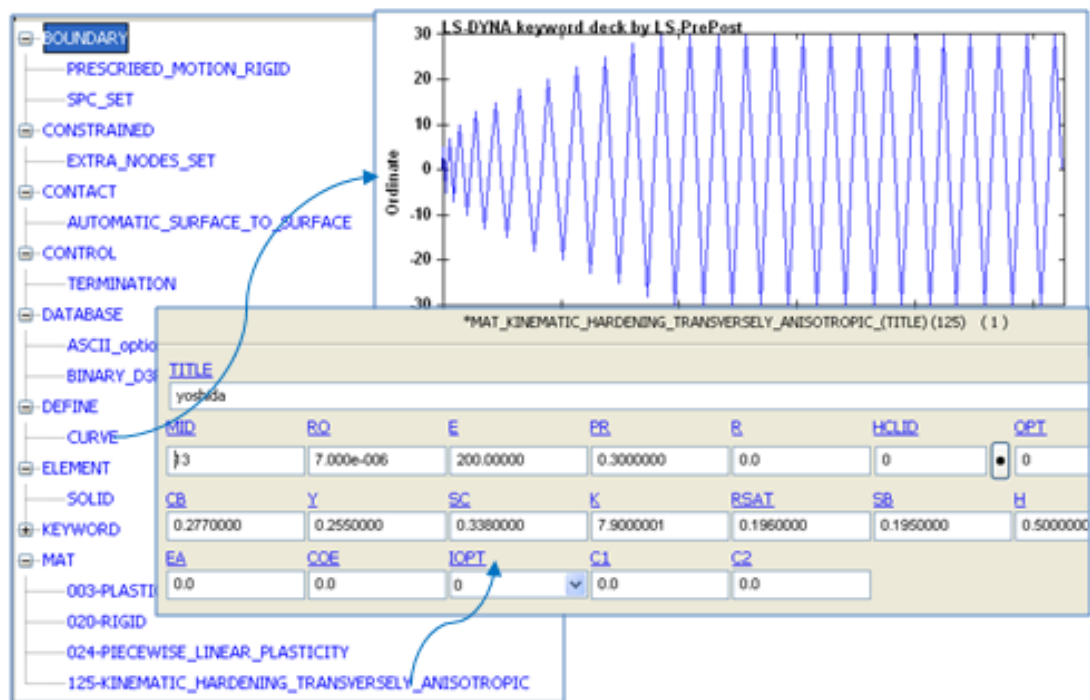


Figure D.2 UFP mini brace – Displacement curve and material properties

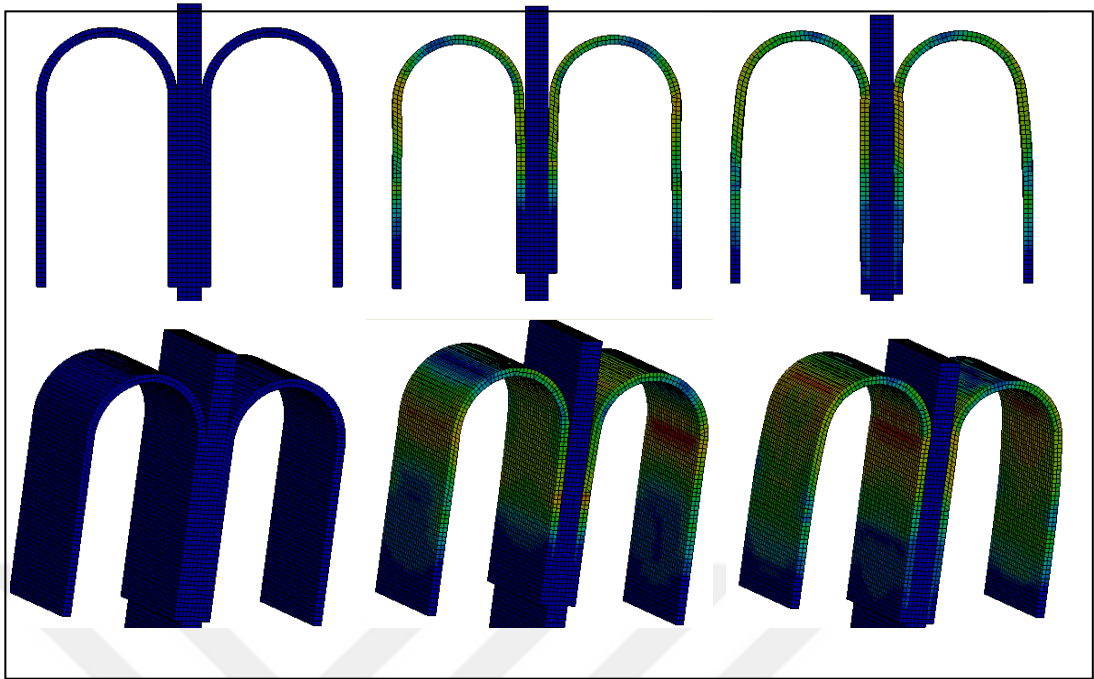


Figure D.3 UFP mini brace – Displacement and Von Misses stresses distribution pattern

APPENDIX E: PRECAST RC FRAME DETAILS

Transverse RC precast frame that was analyzed and retrofitted in this study is presented in Figure E.1. The highlighted sections, namely: the column, beam and the joints are presented in more detail in the subsequent sections. Precast hollow slab panels with the respective connections to form the composite unit are shown in detail as in the current section.

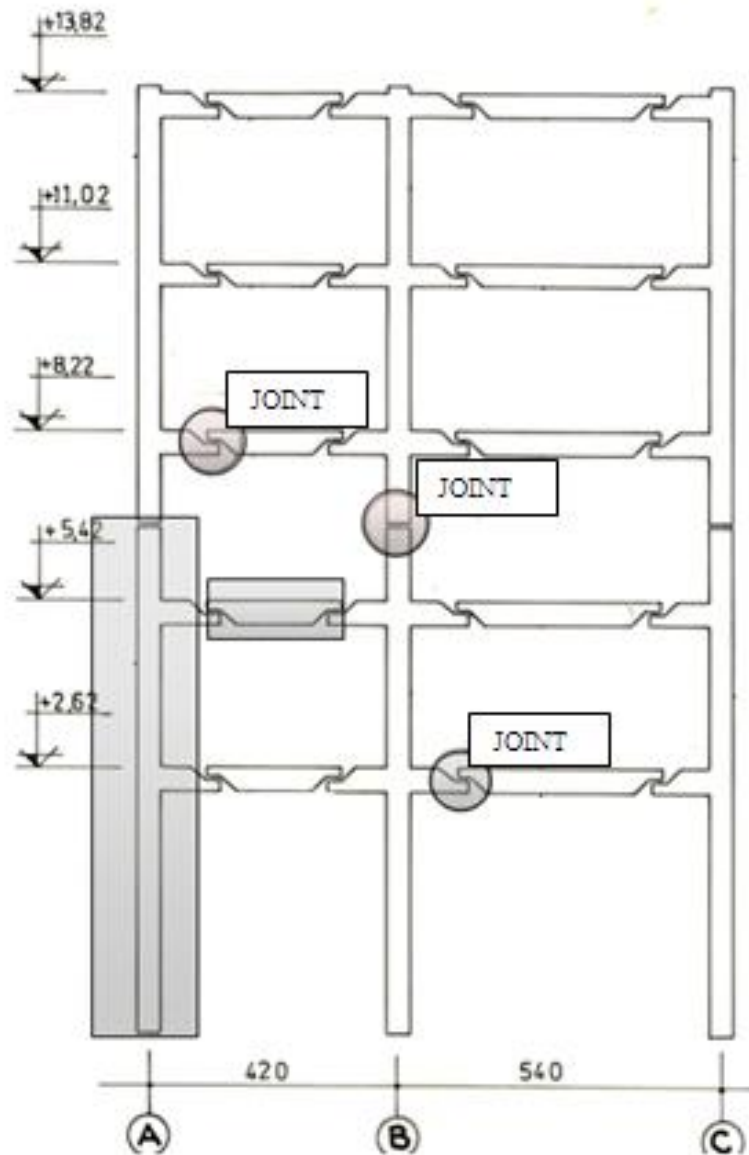
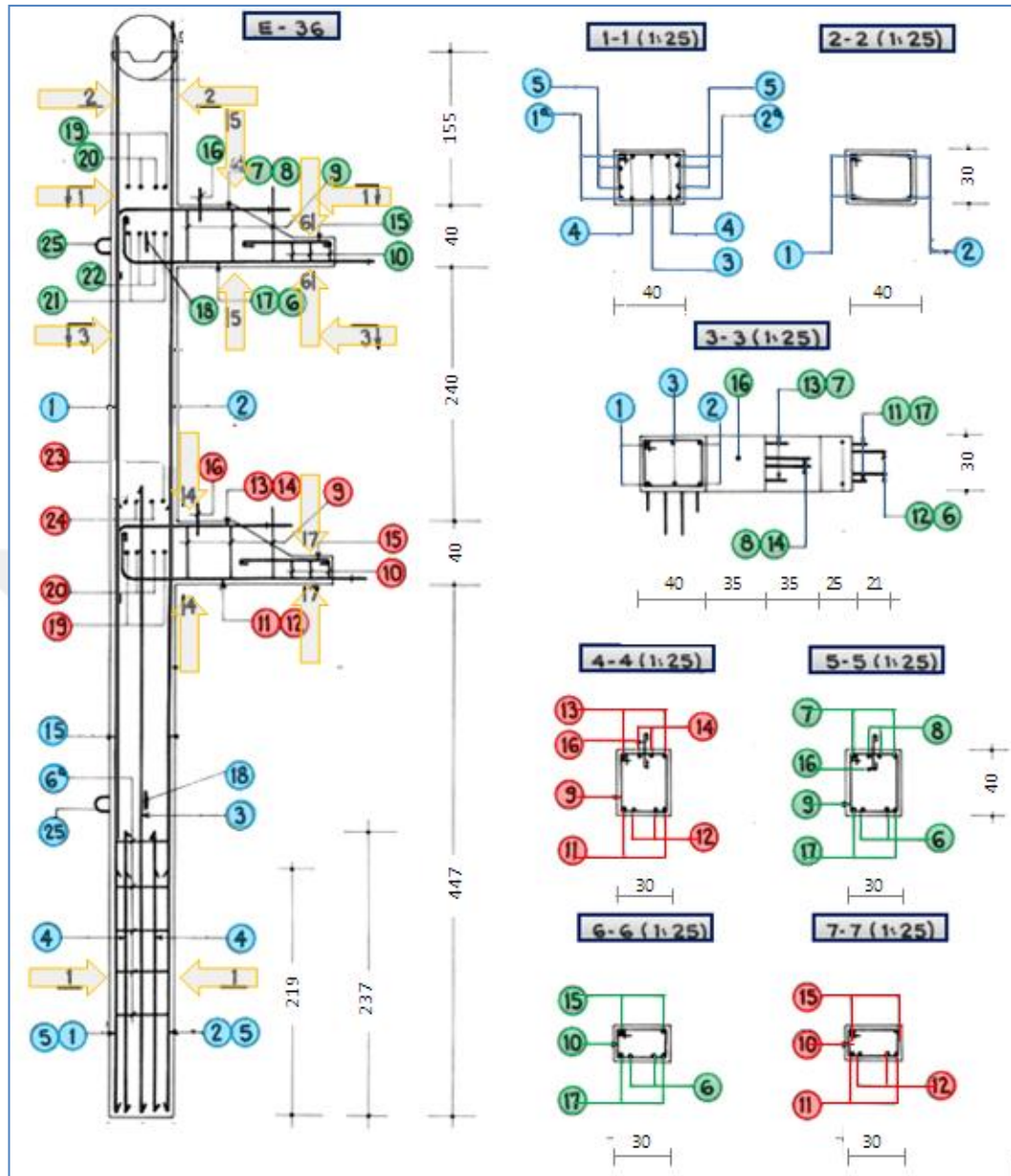
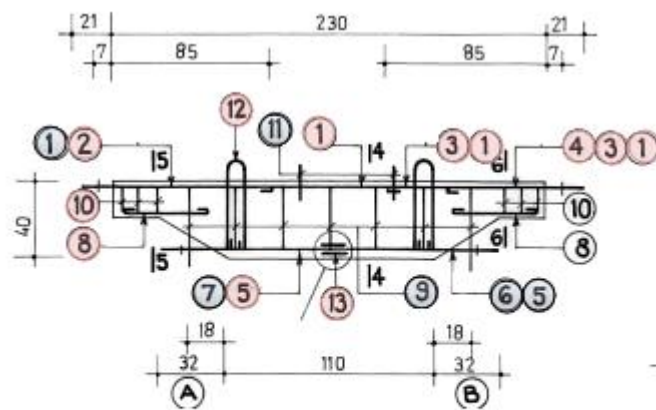


Figure E.1 RC precast frame

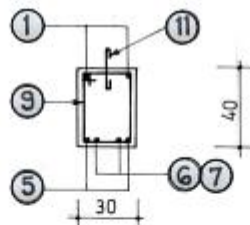


25	16	58	16	43	110
24	26	53	16	43	110
23	24	53	16	43	110
22	18	53	16	43	110
21	18	53	16	43	110
20	22	53	16	43	110
19	22	53	16	43	110
18	16	53	16	43	110
17	16	53	16	43	110
16	10	53	16	43	110
15	12	53	16	43	110
14	24	53	16	43	110
13	24	53	16	43	110
12	18	53	16	43	110
11	20	53	16	43	110
10	6.5	53	16	43	110
9	6.5	53	16	43	110
8	22	53	16	43	110
7	22	53	16	43	110
6	16	53	16	43	110
5	18	53	16	43	110
4	24	53	16	43	110
3	20	53	16	43	110
2	16	53	16	43	110
1	16	53	16	43	110

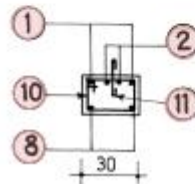
Figure E.2 RC precast column with reinforcement details



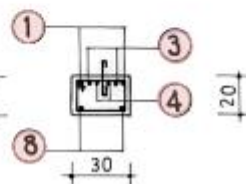
4-4 (1:25)



5-5 (1:25)



6-6 (1:25)



Nr.	φ	Sketch
1	22	272
2	22	92
3	20	92
4	14	76
5	16	174
6	14	133
7	16	133
8	12	48
9	6,5	31
10	6,5	41 26 36 21 26 16
11	10	41
12	8	22 16 41
13	14	14

Figure E.3 RC precast beam with reinforcement details

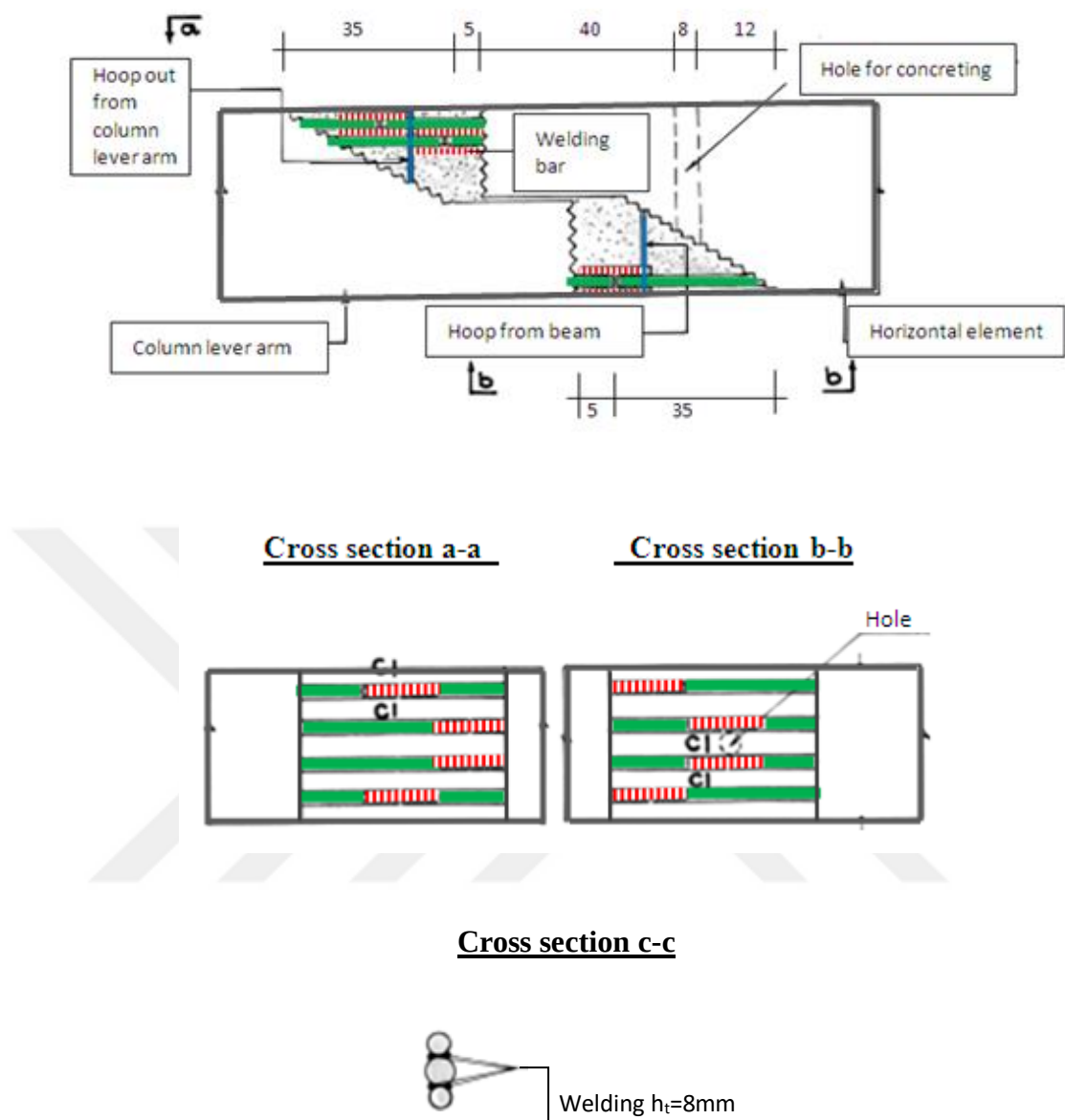


Figure E.4 RC precast beam-column joint detailing

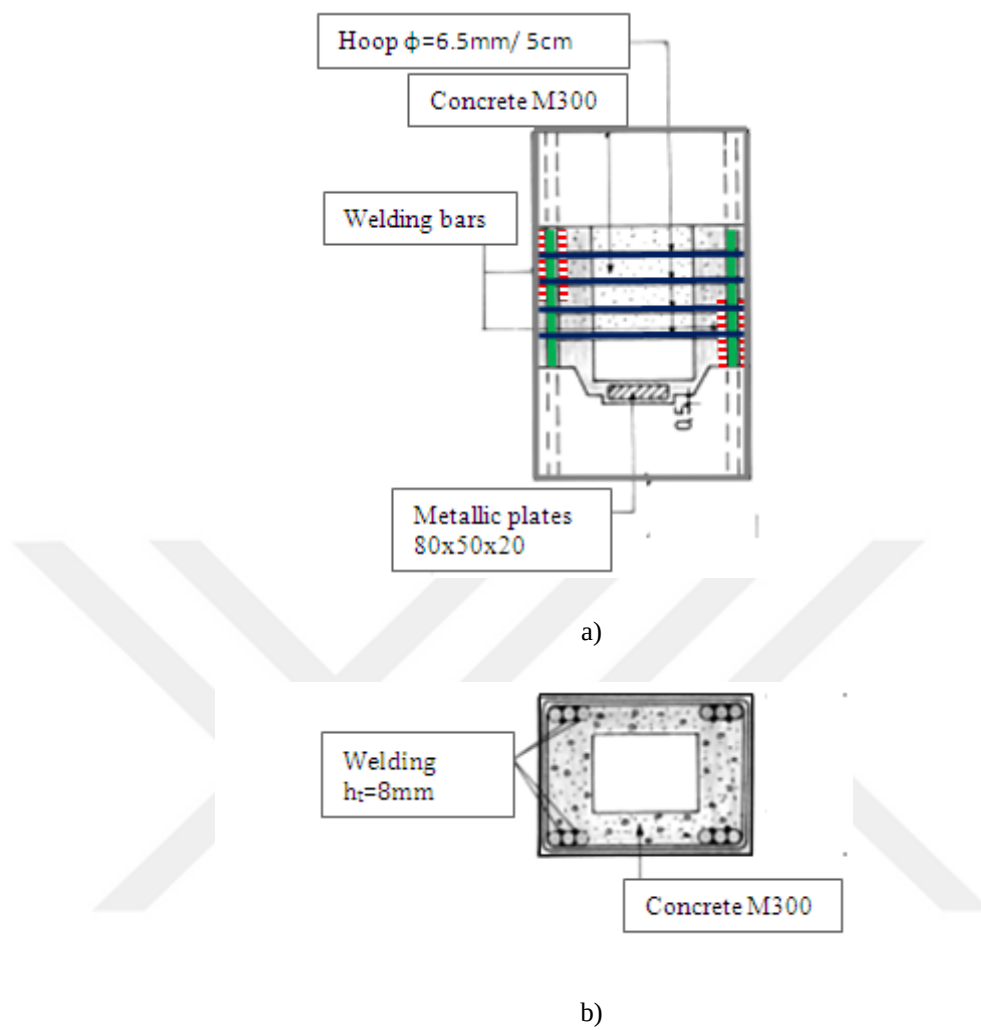


Figure E.5 RC precast column joint detailing a) longitudinal b) transverse cross section

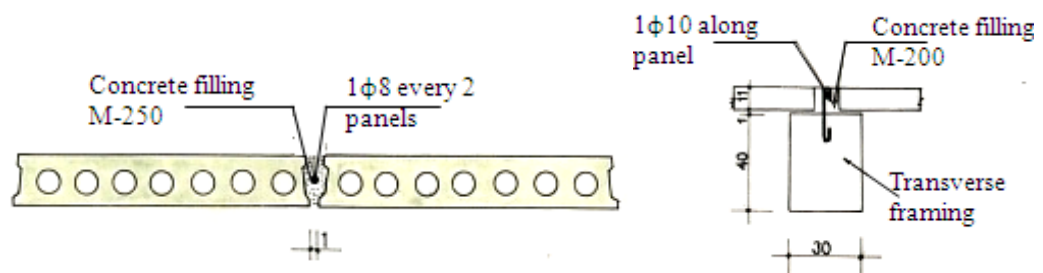


Figure E.6 RC precast hollow panel construction detailing

APPENDIX F: UFP BRACE DETAILS WITH/OUT SELF-CENTERING

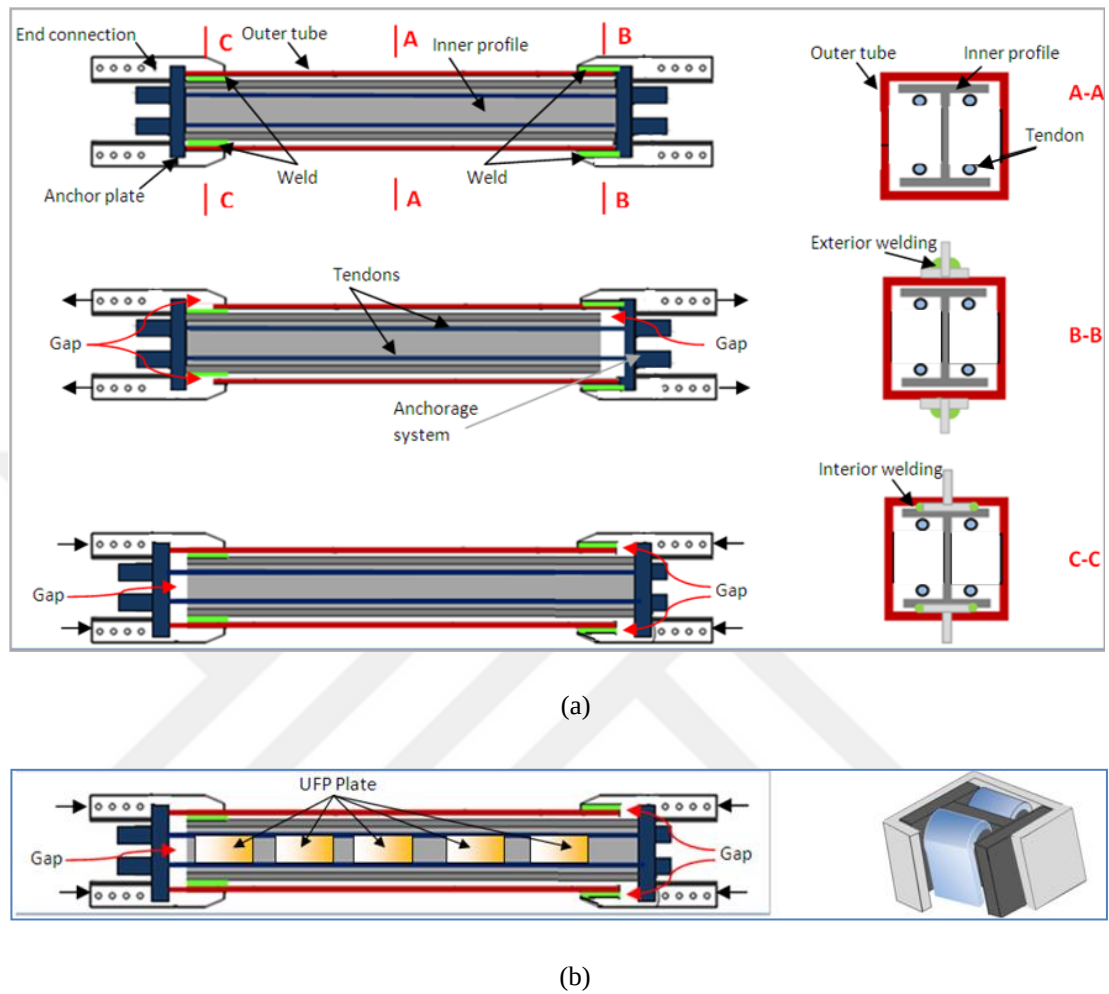


Figure F.1 Schematic representation of brace (a) without UFPs (b) with UFPs with self - centering

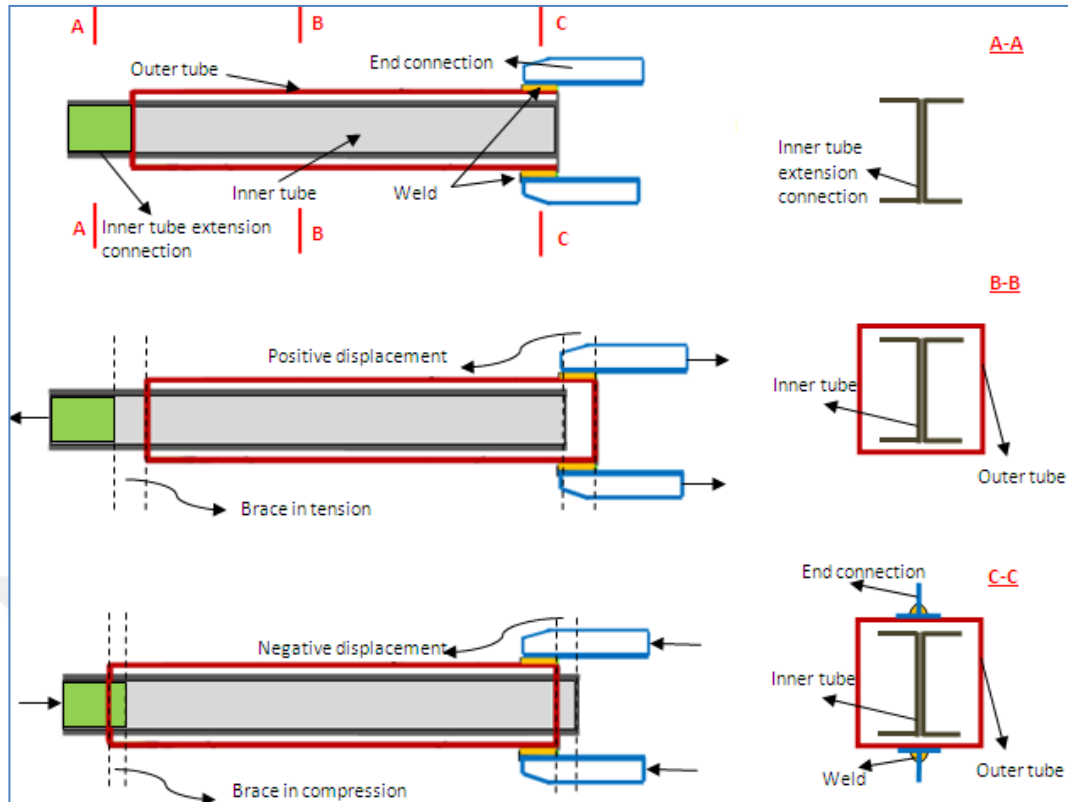


Figure F.2 Schematic representation of brace without UFPs nor self - centering

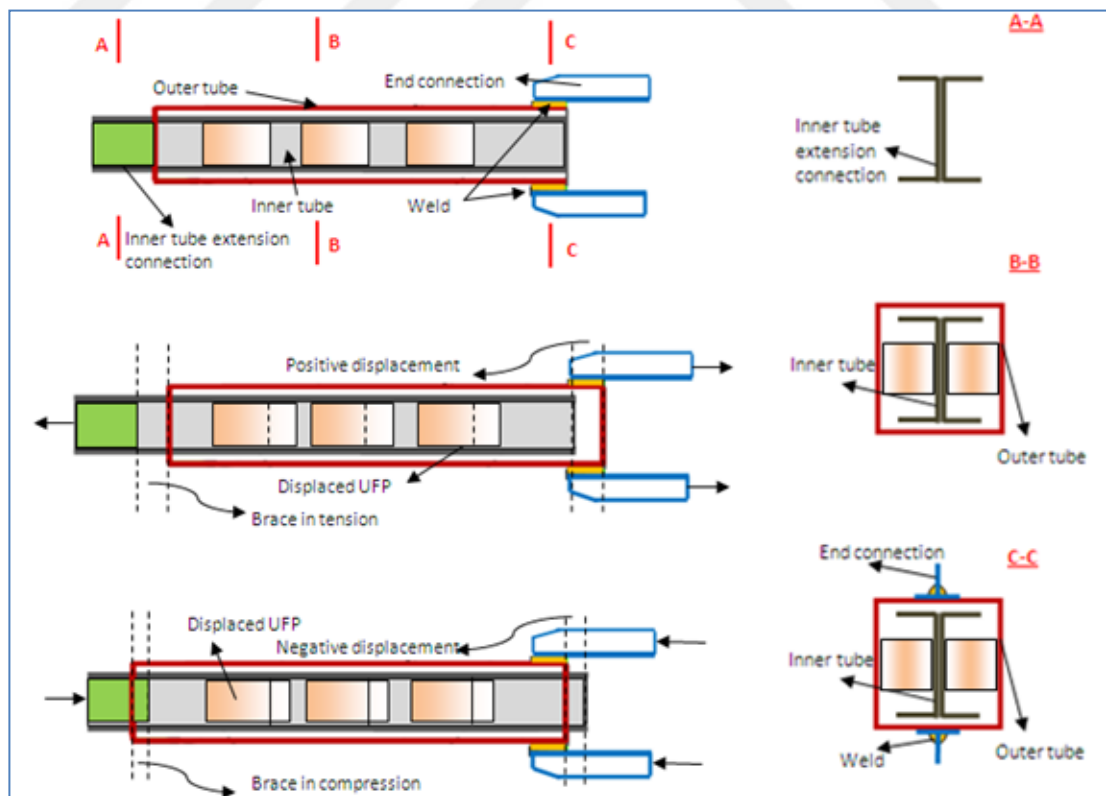


Figure F.3 Schematic representation of brace with UFPs and no self - centering

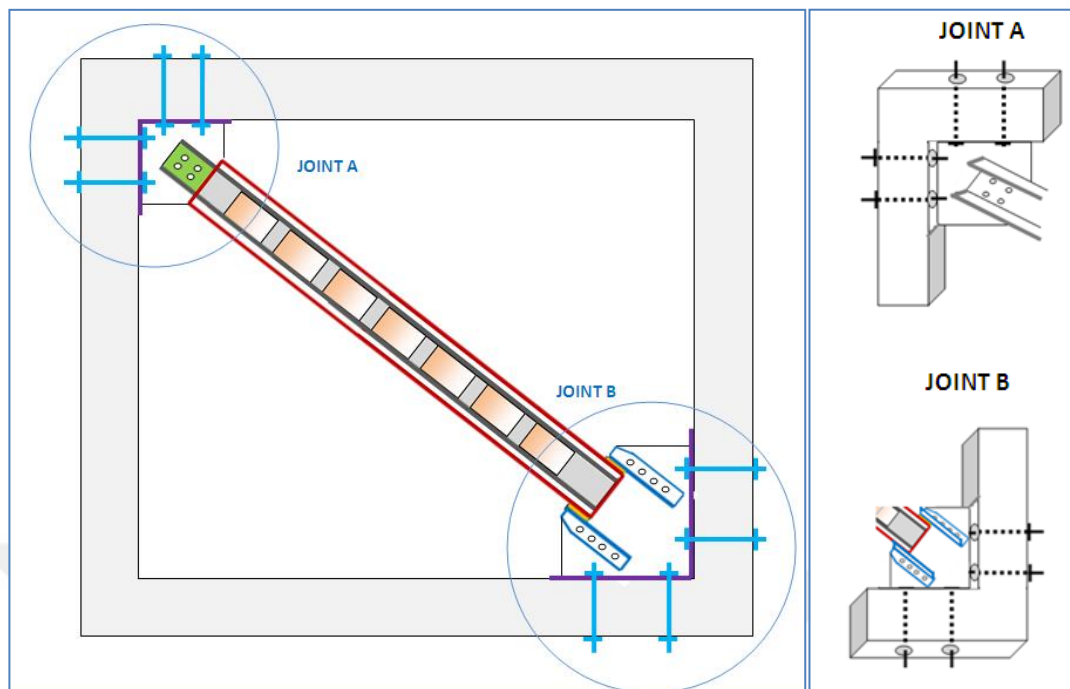


Figure F.4 Schematic representation of brace connection details with RC frame